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Waterways Experiment Station

Corps of Engineers Structural Engineering Conference

8-12 July 1991
St. Johns County, Florida

Volume I

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13. ABSTRACT *(Maximum 200 words)*

This document includes material assembled as a completion report of the Corps of Engineers Structural Engineering Conference (CESEC 91) held in St. Johns County, FL, 8-12 July 1992. Under this cover are the agenda of the conference, abstracts and papers presented, a description of the exhibits, posters and demonstrations displayed, a list of attendees, and an evaluation of the conference by the participants.

The 5-day conference included the presentation of 112 papers, 32 visual displays, six training sessions and two leadership forums on the state-of-the-practice in structural engineering within the Corps of Engineers and other agencies in attendance. The presentations were given as formal talks or informally through visual displays and demonstrations. A challenge workshop was conducted one afternoon where a select group of structural engineers were presented with two emergency situations which could likely occur. These teams of engineers were asked to provide an immediate assessment of the situations and both a short-term and long-term solution to the problems. Leadership forums were conducted for chiefs of structural design from each office for both military and civil works areas.

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The conference was attended by 263 engineers representing 12 division and 36 district offices, HQUSACE, the Construction Engineering Research Laboratory, the Cold Regions Research Laboratory, the Waterways Experiment Station, and non-Corps offices including the Federal Energy Regulatory Commission, Soil Conservation Service, US Bureau of Reclamation, Civil Engineering Research Foundation, Washington University, Lehigh University, Ohio State University, University of Colorado, Intergraph Corp., Black and Veatch, Goldberg & Simpson, and Lester B. Knight & Assoc.

Preface

The Engineering Divisions of the Civil Works and Military Programs Directorates, Headquarters, US Army Corps of Engineers (HQUSACE), sponsored a Corps of Engineers Structural Engineering Conference, 8 - 12 July 1991 in St. John County, FL. The purposes of this conference were to provide for the exchange of structural engineering experience in project design and construction for new projects and the rehabilitation and repair of existing projects, and to review the latest developments in structural engineering research and computer-aided design and drafting.

The conference was attended by 263 engineers representing 12 division and 36 district offices, HQUSACE, the Construction Engineering Research Laboratory, the Cold Regions Research and Engineering Laboratory, the Waterways Experiment Station (WES), and non-Corps offices, including the Federal Energy Regulatory Commission, US Department of Agriculture Soil Conservation Service, US Bureau of Reclamation, Washington University, Lehigh University, Ohio State University, University of Colorado, and Intergraph Corp., Black and Veatch, Goldberg & Simpson, Lester B. Knight & Assoc., and Civil Engineering Research Foundation.

The 5-day conference included the presentation of 112 papers, 32 visual displays, 6 training sessions, and 2 leadership forums on the state-of-the-practice in structural engineering within the Corps of Engineers and the other agencies attending the conference. The presentations were given as formal talks, visual displays, or training workshops. The papers covered both civil works and military projects with a focus on Total Design Quality. A challenge workshop was conducted one afternoon where a select group of structural engineers was presented with two emergency situations which could likely occur. These teams of engineers were asked to provide an immediate assessment of the situations and both a short-term and long-term solution to the problems. Leadership forums were conducted for chiefs of structural design from each office for both military and civil works areas. The conference was planned, organized, and conducted by an HQUSACE-appointed steering committee composed of the following personnel:

Mr. Lucian Guthrie, Structures Branch, CW, HQUSACE, Chairman
Mr. Ray Navidi, Chief, Design Branch, Huntington District
Mr. Don Dressler, Chief, Structures Branch, CW, HQUSACE
Mr. Charles Gutberlet, Technical Engineering Branch, MP, HQUSACE
Ms. Jana Tanner, Structural Section, Jacksonville District
Mr. John Jaeger, Chief, Structural Section, Jacksonville District
Ms. Gina Horri, Structural Section, Jacksonville District
Dr. N. Radhakrishnan, Chief, Information Technology Laboratory (ITL), WES
Mr. Paul K. Senter, Assistant Chief, ITL, WES
Mr. H. Wayne Jones, Chief, Scientific and Engineering Applicatons Center, ITL, WES

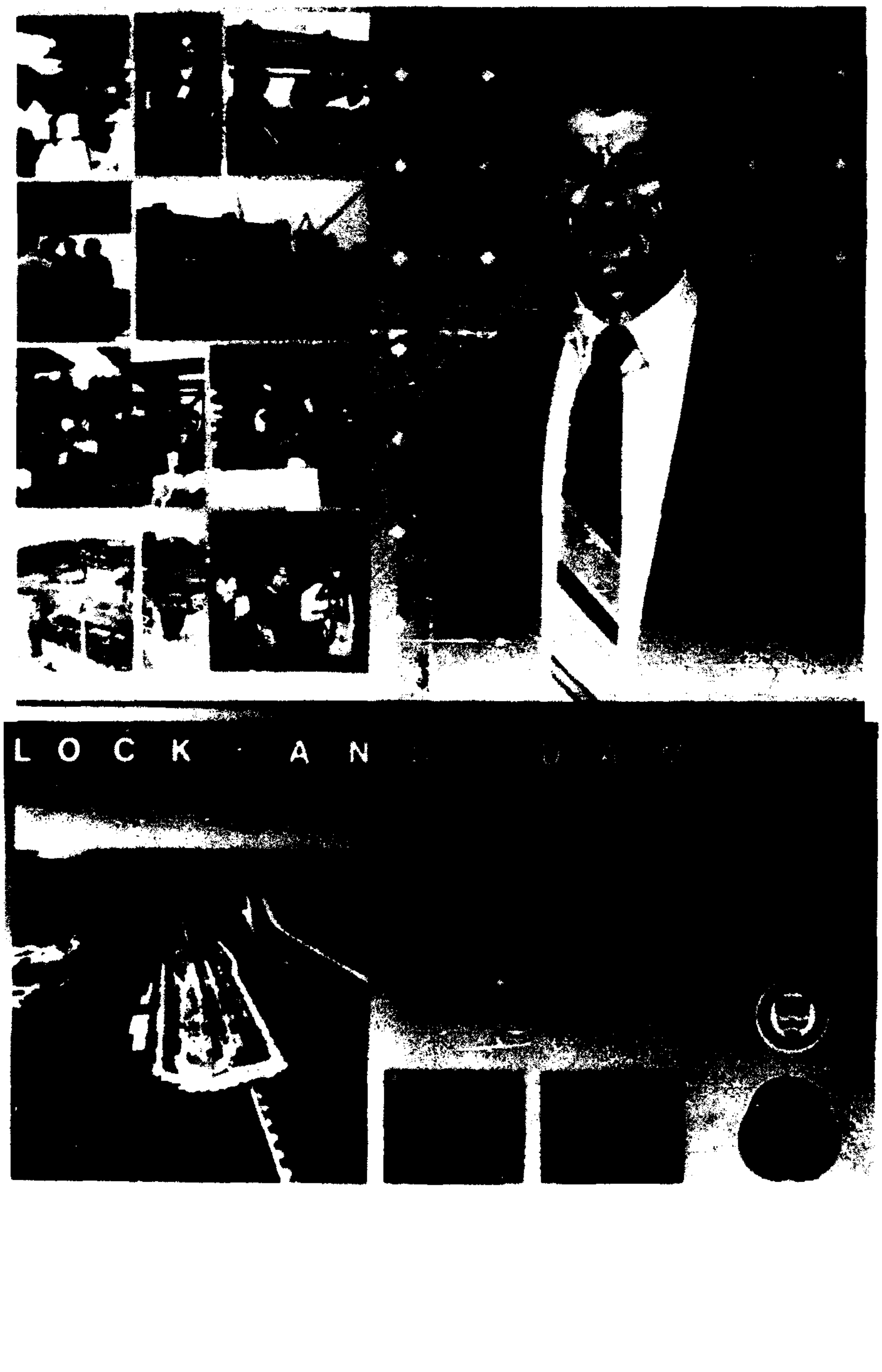
Mr. Guthrie served as the overall conference coordinator and HQUSACE point of contact. Mr. Navidi coordinated the very important work of the conference session chairmen who were responsible for the execution of each session.

Mr. John Jaeger and Ms. Gina Horri coordinated the conference support activities by the Jacksonville District, the largest single job of all. Messrs. Byron Foster, William Wigner, and Joe Hartman coordinated arrangements for the special challenge workshop. Ms. Jana Tanner was responsible for the conference registration. Ms. Vicki Dinkla, Ms. Joy Wells, and Ms. Donzia King provided superior administrative support. Ms. Jo-Ann Guthrie assisted with the registration. Mr. Tony Santana was responsible for the conference audio-visual requirements. Don Cook, Jeffery Couch, Jack Fross, Donna George, Martin Gonzalez, Jim Hawkins, Aaron Kelly, Jim Mangold, Chuck McManus, Steven Robinson, David Shiver, Paul Stroup, and Brent Trauger assisted the conference session chairmen, provided technical support, and helped facilitate all the various conference activities. Mr. Jeff Deemie provided valuable assistance in the development of the conference agenda.

Mr. H. Wayne Jones coordinated the Computer-Aided Structural Engineering (CASE) and visual display sessions and coordinated the publication of this document. Ms. Betty Watson, Visual Information Specialist, ITL, assembled and coordinated the design of this document.

Dedication to Roger Hoell

The Corps of Engineers Structural Engineering Conference, CESEC-91, is dedicated to Roger Hoell, Chief, Structures Section, St. Louis District. Roger was in charge of hosting CESEC-88 and had been a leader in other USACE projects during his career. In 1990 he was stricken with a debilitating disease that forced him to retire prematurely from the St. Louis District. He passed away January 21, 1992.



In Memory of Paul K. Senter

It is only right to remember Paul Senter in this report. He had taken the lead in completing reports of our previous conferences and was working on this one when he became ill.

Paul's untimely death leaves a void in the Information Technology Laboratory at WES and in our hearts that may never be filled. We will not forget his smiling face and his caring nature. You could feel his friendship and depend on it. We have lost a professional associate, but more importantly, we have lost a friend. We give thanks for having known Paul and are better off for it.

We extend our sympathy to Paul's wife, Sarah, and his daughter, Kelly.

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Agenda

Monday - 8 July 1991

General Session 1 — Chairman John Jaeger (SAJ)

7:55 a.m.	Opening Remarks – Edward E. Middleton (SAJ)
8:00 a.m.	Welcoming Remarks – COL Bruce A. Malson (SAJ); Donald R. Dressler (HQ) Conference Overview – Lucian Guthrie (HQ)
8:25 a.m.	Keynote Address – Herbert H. Kennon, Deputy Director of Civil Works
9:05 a.m.	Maintaining Design Quality – Ray Navidi (ORH)
9:35 a.m.	BREAK

Concurrent Sessions

Session 2A Dams — Chairman Carl Betterton (ORD)

9:55 a.m.	RCC Dam Project – Raymond Veselka (SWD)
10:30 a.m.	Olmsted L&D – Holly Gittings, Jeffrey Bayers (ORL)
11:00 a.m.	Portuguese Dam & Monolith Layout & Survey Control – William Wigner (SAJ)

Session 2B Cofferdams & Construction — Chairman Tom Quigley (LMS)

9:55 a.m.	Cofferdam Design Problems – John Gribar (ORP)
10:30 a.m.	Cofferdam Closure – James A. Mills (LMS)
11:00 a.m.	Lateral Movement of L&D Monoliths – Thomas J. Quigley (LMS)

Session 2C Hardened Structures — Chairman Al Knoch (HQ)

9:55 a.m.	Revision of TM's 5-1300 & 5-855-1 – William Gaube (HND), Paul LaHoud (MRO)
10:30 a.m.	Concrete Slabs Subjected to Blasts – Timothy C. Knight (MRO)
11:00 a.m.	Parameters Affecting Response of Slabs to Conventional Weapons Effects – Stanley Woodson (WES)

Session 2D **Buildings — Chairman Richard Maskill (MRD)**

9:55 a.m.	Corps Masonry/Quality Assurance – Ervell Staab (MRD)
10:30 a.m.	Collapse of Tension Fabric Structure – Thomas Wright (MRK)
11:00 a.m.	Wind Damage at Ft. Hood – Joseph Hartman (SWD)
11:30 a.m.	LUNCH

Concurrent Sessions

Session 3A **Structural Problems — Chairman Lucien Mrocziewicz (NPD)**

12:55 p.m.	Structural Problems in Flat Slabs Foundation Ties and Harbor Improvements – Arthur Shak (POD)
1:30 p.m.	Fish Passage Structures – David Raisanen (NPD)
2:00 p.m.	Fish Collection Facility, Problems – Bruce McCracken (NPP)

Session 3B **Flood Control — Chairman Byron Foster (SAD)**

12:55 p.m.	Passaic River Flood Control Project – Douglas Pendrell (NAN)
1:30 p.m.	Puerto Rico Flood Control Project – Gina Horri (SAJ)
2:00 p.m.	St. Johns River Basin Flood Control Project – Charles B. McManus (SAJ)

Session 3C **Hardened Structures — Chairman Al Knoch (HQ)**

12:55 p.m.	Blast Response of Unreinforced Masonry Walls with Openings – David R. Coltharp (WES)
1:30 p.m.	Nonlinear FEA Applied to Blast Resistant Structure – Bruce Walton (MRO)
2:00 p.m.	In-Structure Shock & Shock Isolation – William Seipel II (MRO)

Session 3D **Gates & Weldment — Chairman Tom Mudd (WES)**

12:55 p.m.	Roller Gate Design – Thomas J. Wirtz (NRC)
1:30 p.m.	Weldment Residual Stresses – John Jaeger (DAJ)
2:00 p.m.	Sillway Trash Gate – Michael Thompson (SAM)
2:30 p.m.	BREAK

Concurrent Sessions

Session 4A — Locks — Chairman Gordon McClellan (ORN)

2:55 p.m.	Slurry Constructed Diaphragm Guard Wall – Jerome A. Maurseth, Jeffrey S. Sedey (NPP)
3:30 p.m.	McAlpine Lock Replacement – Veronica Rife (ORL)

Session 4B — Tunnels — Chairman Hari Singh (NCD)

2:55 p.m.	San Antonio Flood Control Tunnels – William A. Wallace (SWF)
3:30 p.m.	Bassett Creek Flood Control Tunnel – Bruce Brand (NCS)

Session 4C — Hardened Structures — Chairman Al Knoch (HQ)

2:55 p.m.	In-Structure Shock Calculation Buried Structures – Richard Dove (WES)
3:30 p.m.	Airblast Attenuation Devices – Randy Holmes (WES)

Session 4D — Closures Gates — Chairman Carl Betterton (ORD)

2:55 p.m.	St. Peter Flood Gates – Alan Schulz (LMS)
3:30 p.m.	Precast Seal Beam Flood Control Closure – James Gunnels (ORN)
4:00 p.m.	ADJOURNMENT
5:00 p.m. -7:00 p.m.	ICEBREAKER

Tuesday – 9 July 1991

7:55 a.m.	ANNOUNCEMENTS

Separate Session — Military Programs Structural Chiefs Meeting

General Session 1 — Chairman Lucian Guthrie (HQ)

8:00 a.m.	Quality in the Constructed Project: ASCE Guide – Stephen C. Mitchell, Lester B. Knight & Associates
8:40 a.m.	Structural Engineering within Civil Works — Direction and Priorities – Don R. Dressler (HQ)
9:10 a.m.	Sheet Pile Walls, Developments – Reed Mosher (WES)

9:40 a.m.	BREAK

Session 2A — Tie Back Walls — Chairman John Gribar (ORP)

9:55 a.m.	Unique Soil Tie Back Wall – John Hager (SAS)
10:30 a.m.	Design, Construction & Evaluation of Earth Anchors, – John Grundstrom, Ron Erickson (NCE)
11:00 a.m.	Temporary Tie Back Wall, Bonneville – John Etzel (NPP)

Session 2B — Outlet Works & Cutoff Wall — Chairman Garrett Johnson (NPS)

9:55 a.m.	Mudd Mountain Dam Intake/Outlet Works – Paul C. Noyes (NPS)
10:30 a.m.	Mudd Mountain Dam Cutoff Wall Deformation Analysis – E. Wayne Kutch (NPS)
11:00 a.m.	Seven Oaks Dam Outlet Works – Raymond R. Dewey (NPP)

Session 2C — Spillway & Miter Gates — Chairman Bruce Riley (ORP)

9:55 a.m.	Tainter Gate Analysis – David J. Smith (MRO)
10:30 a.m.	Submersible Gate Wicket Replacement – David R. Wehrley (NCR)
11:00 a.m.	FEA of Miter Gate Anchorage – Bruce Riley (ORP)
11:30 a.m.	LUNCH

Session 3A — Structural Damage, Restoration, and Design — Chairman Dave Descoteaux (NED)

12:55 p.m.	Structural Damages from Hurricane Hugo – Rick Lambert (SAC)
1:30 p.m.	Structural Aspects of Superfund Site Design – William Strobach (MRK)
2:00 p.m.	Ben Sawyer Bridge Restoration – Mark Nelson (SAC)

Session 3B — Flood Control Projects — Chairman Terry Cox (LMV)

12:55 p.m.	Blue River Paved Research – Morris Ganaden (MRK)
1:30 p.m.	U-Flume Channel Design – Kurt Mitscher (MRK)
2:00 p.m.	Butterfly Valve Structure – Walter O. Baumy (LMN)

Session 3C — Miter Gates — Chairman Stacy Anastos (NAD)

12:55 p.m.	Miter Gate Diagonals, Stressing and Fatigue Concerns – Thomas J. Leicht (HQ)

1:30 p.m.	Barge Impact Testing on Miter Gates – Thomas Ruf (LMS), John Jaeger (SAJ)
2:00 p.m.	Miter Gate Sill Repair Caisson – Ralph Snowberger (ORL)
2:30 p.m.	BREAK

Session 4A Bridges — Chairman Arthur Shak (POD)

2:55 p.m.	Rehabilitation of Timber Bridge – Lucy Ortiz (SWA)
3:30 p.m.	TBA

Session 4B Nonlinear, Incremental, Structural Analysis (NISA) — Chairman Wayne Jones (WES)

2:55 p.m.	NISA and Fracture Mechanics – Barry D. Fehl (LMS)
3:30 p.m.	NISA of Olmsted Locks – Chris Merrill, Sharon Garner (WES)

Session 4C Erosion Control Project & Pipe Repairs — Chairman Arvis Dennis (LMK)

2:55 p.m.	Erosion Control Project, Yazoo Basin – Jonathan W. Trest (LMK)
3:30 p.m.	84" CMP Repair – Tamara Atchley (LMS)
4:00 p.m.	ADJOURNMENT

Wednesday - 10 July 1991

General Session 1 — Chairman Charles Gutberlet (HQ)

7:55 a.m.	ANNOUNCEMENTS
8:00 a.m.	Responsibility for Design of Steel Structures – David B. Ratterman, Goldberg and Simpson
8:40 a.m.	High Strength Bolted Connections - Clarifying Issues – Ray Decker (MRD)
9:10 a.m.	Computer-Aided Engineering – Dr. N. Radhakrishnan and Paul K. Senter (WES)
9:40 a.m.	BREAK

Session 2A R&D and SSI — Chairman Victor Agostinelli (LMV)

9:55 a.m.	SSI Analysis of U-Frame – Reed Mosher, Robert Ebeling (WES)
10:30 a.m.	Retaining Wall Analysis R&D – Reed Mosher (WES)
11:00 a.m.	Anchored Steel Pile Bulkhead Analysis – Reed Mosher, Kevin Abraham (WES)

Session 2B	Channel Walls and Bulkhead — Chairman Joe Keith (ORL)
9:55 a.m.	Post & Panel Wall with Anchors – Daniel E. Beyke (SAS)
10:30 a.m.	Bulkhead Designs & Construction Problems – Kirti Joshi (SAS)
11:00 a.m.	Channel Wall Study – Rochelle Ross, Tamara Atchley (LMS)

Session 2C	Buildings — Chairman Larry Seals (ORD)
9:55 a.m.	Investigation & Repair, AFB Hospital – George Diewald (SWA)
10:30 a.m.	Rehabilitation of Buildings, AFB – Ted M. Solano (SWA)
11:00 a.m.	Child Development Centers, AF – Eric J. Fry (ORL)

Session 2D	Buildings — Chairman Paul LaHoud (HND)
9:55 a.m.	Solid Rocket Motor Assembly Building – Vira Khim (SAM)
10:30 a.m.	Unit Chapel, Ft. Campbell – Jeffrey Bayers (ORL)
11:00 a.m.	Interface Friction Effects on Buried Arches – Frank Dallriva (WES)
11:30 a.m.	LUNCH
1:00 p.m.	**Challenge Workshop (Preassigned Attendance)**

Visual Display Presentations

- Performance of Rebar Locating Device (WES)
- RIGID Computer Program for Dynamic Analysis of Rigid Bodies (MRO)
- BLASTX Computer Program for Airblast Effects from Interior Explosions (MRO)
- Nonlinear Finite Element Analysis Exhibit (MRO)
- SRC Computer Program for Shock Spectra Response Calculations
- Expected Stresses in Dolo Armor Units (WES)
- Strength Development of Concrete Cured at Low Temperatures
- Condition Assessment, Steel & Concrete Cured at Low Temperatures (CERL)
- Improved Strength, Design, Hydraulic Structures (WES)
- Penstock Replacement (MRO)
- Underwater Repair (MRO)
- Gathright Tower Vibrations (WES)
- Barge Impact Test Video
- Fracture Analysis - Concrete Hydraulic Structures
- Metrification of Construction
- Statistical Analysis System
- CASE Arch Dam Workstation
- CASM - Computer Aided Structural Modeling
- Conversationally Oriented Real-Time Programming Library

- Mesh Generation on CADD Workstation
- Computer-Aided Structural Engineering Program Demos
- Nonlinear Incremental Structural Analysis—NISA (WES)
- Portuguese Arch Dam Model (SAJ)
- Rio Puerto Nuevo - CADD Application (SAJ)
- Civil Engineering Research Foundation

4:00 p.m. ADJOURNMENT

6:00 p.m. -9:00 p.m. BARBEQUE

Thursday – 11 July 1991

General Session 1 — Chairman Tom Leicht (HQ)

7:55 a.m. ANNOUNCEMENTS

8:00 a.m. CW Guidance Update Program – Tom Mudd (WES)

8:30 a.m. Earthquake Implications for Central & Eastern U.S. – Helen Peterson (MRK)

9:00 a.m. Bridge Fatigue Analysis – Donald Logsdon (NCR)

9:30 a.m. BREAK

Session 2A Dam Safety — Chairman Brent Trauger (SAJ)

9:55 a.m. COE Dam Safety Program – John McPherson (HQ)

10:30 a.m. Anchoring of Kerr Dam – Christy L. Hannan (SAW)

11:00 a.m. Folsom Dam Seismic Evaluation – John Nickell (SPK), Dr. Robert Hall (WES)

Session 2B Seismic Design - Dams & Towers — Chairman Paul Senter (WES)

9:55 a.m. Non-linear Dynamic Analysis of Arch Dam – James Mangold (SAJ)

10:30 a.m. Non-linear Response of Gravity Dams – Dr. Robert Hall (WES)

11:00 a.m. Seismic Evaluations of Intake Towers – David Descoteaux (NED)

Session 2C Building Design — Chairman William Wallace (SWF)

9:55 a.m. Vibro-Acoustic Study of Aircraft Maintenance Dock – James Wilcoski, Louis Sutherland (CERL)

10:30 a.m. Non-destructive Evaluation of Masonry – Robin Westerfield (SWF)

11:00 a.m.	Dynamic Testing & Analysis of Radar Facility – Joseph M. Serena, William H. Zehrt, Jr., Arthur G. Dohrman (MRK)

Session 2D **Seismic Design Buildings, Chairman John Hayes (CER)**

9:55 a.m.	Seismic Design for Weapons Facilities – Roy Stephen Wright (HND)
10:30 a.m.	Steel Deck Diaphragm Design Methods – Christopher G. Glatt (MRK)
11:00 a.m.	Seismic Engineering Research for MP & CW – John R. Hayes, Dr. Robert Hall (CERL & WES)

Session 3A **Rehabilitation Repairs and R&D — Chairman Don Berger (SPD)**

12:55 p.m.	REMR/CPAR – W. E. Roper (HQ)
1:30 p.m.	Condition Evaluation and Analysis – Dr. Mary Leggett (WES)
2:00 p.m.	Lift Gate Failure Lock 27 – Robert Kelsey (LMS)

Session 3B **Seismic Design and Rehabilitation — Chairman Jim Tonouye (SPD)**

12:55 p.m.	Seismic Analysis of Olmstead Locks – Dr. Robert Hall and Tommy Bevins (WES)
1:30 p.m.	Model for Seismic Analysis of Pile Groups – Reed Mosher and Robert Ebeling (WES)
2:00 p.m.	Rehabilitation of Eisenhower and Snell Locks – Reed Mosher (WES)

Session 3C **Building Design — Chairman George Diewald (SWA)**

12:55 p.m.	Structural Reliability & Impact on Design – Nathan M. Kathir (NCS)
1:30 p.m.	Lateral Stability of Beams – Bruce Brand (NCS)
2:00 p.m.	Automated Modular Design (Kit-of-Parts) – Anjana Chudgar (ORD)

Session 3D **Seismic Design - Buildings — Chairman Robert Hollendeck (NPW)**

12:55 p.m.	Vulnerability & Upgrading of Nonductile Concrete Frames – Pamalee Brady (CERL)
1:30 p.m.	Testing of Base Isolator – James B. Gambell (CERL)
2:00 p.m.	Masonry Program Development – Hal C. Thomas (SAS)
2:30 p.m.	BREAK

Session 4A **Fracture Analysis — Chairman Ervell Staab (MRD)**

2:55 p.m.	Fracture Analysis of Lock Wall, – Prof. Victor Saouma
3:30 p.m.	Fracture Toughness of 3" MSA Concrete – James N. Ryan (NPS)

Session 4B **Rehabilitation — Chairwoman Gina Horri (SAJ)**

2:55 p.m. Reanalysis & Rehabilitation of Black Rock Lock – Eugene N. Lenhardt (NCB)

3:30 p.m. Evaluation & Rehabilitation of Lock Walls – Munther Sahawneh (SAM)

Session 4C **Evaluation and Dam Safety — Chairwoman Anjana Chudgar (ORD)**

2:55 p.m. Evaluation of Dam Pier Cracks – Haskell Wright (SWL)

3:30 p.m. Design of Training Wall Extension – Richard Shanks (MRK)

Session 4D **Urban Search, Rescue & Evaluation — Chairwoman Pamalee Brady**

2:55 p.m. Engineers Role in Urban Search & Rescue – Ed Hecker (SPD) and David Hammond (Hammond Eng.)

3:30 p.m. The Corps of Engineers and ATC-20 – Jim Tonouye (SPD) and Jim Couey (SPK)

Friday – 12 July 1991

General Session 1 – Chairman Byron Foster (SAD)

7:55 a.m. ANNOUNCEMENTS

8:00 a.m. Challenge Workshop Report – Joe Hartman (SWD), William Wigner (SAJ)

9:00 a.m. Quality Facility Data: Cradle to Grave – Ronald L. Hollrah, Black & Veatch

9:40 a.m. BREAK

General Session 2

10:00 a.m. PANEL SESSION: Total Design Quality – Chairman, George Gibson

11:00 a.m. CONFERENCE SUMMARY

11:20 a.m. CLOSING REMARKS

11:30 a.m. LUNCH

1:00 p.m. **CIVIL WORKS LEADERSHIP FORUM**, Donald R. Dressler (HQ)

Concurrent Training Sessions

Session 1 **SSI Analysis of Sheet Pile Structures Including Soil Anchors**

Session 2 **Nonlinear, Incremental Stress Analysis (NISA)**

Session 3	**Seismic Design & Evaluation of Intake Towers**
Session 4	**Fitness for Service, Bridges & Gates**
Session 5	**Winc Load Design**
Session 6	**Masonry Design**
Session 7	**Hardened Structures Design**

4:00 p.m. ADJOURNMENT

CESEC 91 Papers/Abstracts

Monday—8 July 1991

Keynote Address – Herbert H. Kennon, Deputy Director of Civil Works

Conference Photographs

Maintaining Design Quality in the Corps of Engineers – G. Ray Navidi, Chief, Design Branch, US Army Engineer District, Huntington

Expedited Design and Review Procedures Used for Cuchillo Negro Dam, Truth or Consequences, NM – Raymond Veslka, Structural Engineer, US Army Engineer Division, Southwestern

Olmsted Locks and Dam – Holly A. Gittings, Structural Engineer and Project Coordinator, Engineering Division, US Army Engineer District, Louisville, and Jeffrey E. Bayers, Structural Engineer, Engineering Division, US Army Engineer District, Louisville

Portugues Dam Monolith Layout and Survey Control – William K. Wigner, Jr., Structural Engineer, US Army Engineer District, Jacksonville

Cofferdam Design Problems, Point Marion Lock, Monongahela River, Pennsylvania – John A. Gribar, Chief, Design Branch, Engineering Division, US Army Engineer District, Pittsburgh

Melvin Price Locks and Dam, Third Stage Temporary Closure – James A. Mills, Structural Section, US Army Engineer District, St. Louis

Melvin Price Locks and Dam Lateral Movements of Monoliths – Richard R. Sovar and Thomas J. Quigley, US Army Engineer District, St. Louis

Criteria Update Related to TM 5-855-1 – William H. Gaube, Protective Design Center, US Army Engineer District, Omaha

Revision of the Tri-Services Design Manual Structures to Resist the Effects of Accidental Explosions (TM 5-1300, NAVFAC P-397, AFM 88-22) – Paul M. LaHoud, US Army Engineer Division, Huntsville

Design and Behavior of Concrete Slabs Subject to Blast – Tim Knight, Protective Design Center, US Army Engineer District, Omaha

Parameters Affecting the Response of Slabs to Conventional Weapons Effects – Stanley C. Woodson, Structures Laboratory, US Army Engineer Waterways Experiment Station

Corps Masonry/Quality Assurance – Ervell A. Staab, Chief, Architectural and Structural Branch, US Army Engineer Division, Missouri River

Collapse of a Long-Span Tensioned Fabric Structure – Thomas D. Wright, Structures Section, US Army Engineer District, Kansas City

Wind Damage at Fort Hood – Joseph P. Hartman, Structural Branch, US Army Engineer Division, Southwestern

Structural Problems in the Pacific Ocean Division – Arthur T. S. Shak, Structural Engineer, US Army Engineer Division, Pacific Ocean

Columbia River Fish Passage Structures – David C. Raisanen, Hydroelectric Design Center, US Army Engineer Division, North Pacific

The North Fork Toutle River Fish Collection Facility Design and Construction – Bruce H. McCracken, Structural Engineer, US Army Engineer District, Portland

Passaic River Flood Protection Project – Douglas J. Pendrell, Chief, Geotechnical Design Branch, Passaic River Division, US Army Engineer District, New York

Rio Puerto Nuevo Flood Control Project, San Juan, Puerto Rico – Gina M. Horri, Structural Engineer, US Army Engineer District, Jacksonville

Upper St. Johns River Basin Project Flood Control Structures – Charles B. McManus, Structural Engineer, US Army Engineer District, Jacksonville

Penetration Resistance and Blast Response of Masonry Walls – David R. Coltharp, Structural Engineer, US Army Engineer Waterways Experiment Station

Nonlinear Finite Element Analysis Applied to Blast-Resistant Structures – Bruce A. Walton, Protective Design Center, US Army Engineer District, Omaha

In-Structure Shock and Shock Isolation – William A. Seipel II, Protective Design Center, US Army Engineer District, Omaha

Gallipolis Lock and Dam Roller Gate Design – Thomas J. Wirtz and Carl H. Johnson, Structural Engineers, US Army Engineer District, Rock Island

Determination of Residual Stress and Effects in Thick Section Weldments for Hydraulic Structures – Chon L. Tsai, Professor, Welding Engineering Department, Ohio State University, Columbus, OH; John J. Jaeger, Structural Engineer, US Army Engineer District, Jacksonville; and Jeffrey S. Sedey, Structural Engineer, US Army Engineer District, Portland

Robert F. Henry Spillway Trash Gates – Michael D. Thompson, Engineering Division, Structural and General Engineering Branch, US Army Engineer District, Mobile

Slurry Constructed Diaphragm Guard Wall, Bonneville Navigation Lock – Jerome Maurseth and Jeffrey S. Sedey, Structural Engineers, US Army Engineer District, Portland

McAlpine Lock Replacement Project, McAlpine Locks and Dam – Veronica L. Rife, Structural Engineer, US Army Engineer District, Louisville

Construction of the San Antonio Flood Tunnels – William A. Wallace, Structural Engineer/ Project Leader, US Army Engineer District, Fort Worth

Bassett Creek Tunnel Flood Control Project – Thomas B. Sully, Structural Engineer, US Army Engineer District, St. Paul

In-Structure Shock Prediction for Buried Structures – Structures Laboratory, US Army Engineer Waterways Experiment Station

Effectiveness of Passive Airblast Attenuation Devices Against Conventional Weapons Effects – Randy L. Holmes, Structural Engineer, US Army Engineer Waterways Experiment Station

Architectural and Design Features of the St. Peter Street Floodgates, New Orleans, Louisiana: A High Profile Project – Alan D. Schultz, Structural Design Section, US Army Enginer District, New Orleans

Precast Seal Beam for Railroad Closures – James Gunnels, Structural Engineer, US Army Engineer District, Nashville

Keynote Address

Challenge of the 90's – Change

by
Herbert H. Kennon[1]

- Reduce capacity
- Reduce costs
- Increase quality
  - Do we meet our customers requirements at a fair cost?
- Increase responsiveness
  - Time, costs, quality
  - Roles and responsibilities need redefining
- Adjust product line
- Improve customer service
  - Automation
    CADSE conference
    CADD Center
  - Criteria
- Maintaining competence
- Better distribution of workload
- Flexibility
- Look at quality of life and reducing costs
- Workload driven
- 36 to 22 districts
- 6600 personnel moves
- BRAC – CW committees
- Working together (examples)

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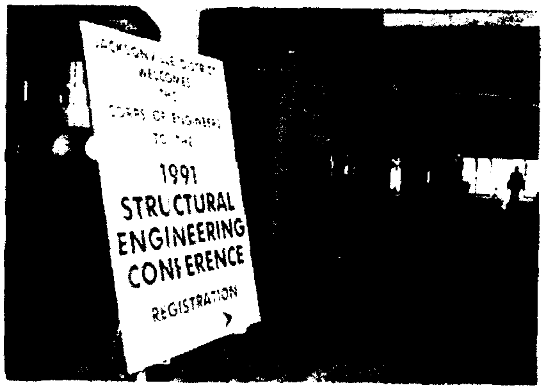
[1] Deputy Director of Civil Works.

CESEC 91 Conference



General Sessions

Parallel Speciality Sessions

Panel Discussions

Challenge Workshop

Ice Breaker

Support Staff

Maintaining Design Quality in the Corps of Engineers

by
G. Ray Navidi[1]

Abstract

There is a concern among the technical staff that the Corps' design products in the future may be of lower quality than they have been to date. In preparation for this paper, a questionnaire on design quality was prepared and sent to the Chiefs of Structural Sections of the Corps Districts. Responses were received from not only the Structural discipline, but also from the other disciplines within Engineering Division. Results of the questionnaire are tabulated in the paper along with a discussion of design quality in engineering.

Introduction

Changes within the past few years in the organization of the Corps of Engineers have had considerable impact on the agency's traditional way of planning, designing, and constructing the Civil Works projects. While these changes are aimed for completing projects on time and within budget, design quality, which has been the Corps' hallmark, seems to have been overlooked.

As a 17-year employee of the Corps who has a high regard for the organization and its accomplishments, the writer has been concerned about a gradual decline of emphasis on maintaining design quality in the Corps of Engineers. Informal discussions over the years with colleagues throughout the Corps have indicated that they, too, have the same concern. In preparation for this paper, a questionnaire on design quality was developed and sent to the Structural Chiefs of 37 Corps Districts which represent all Districts in CONUS. Inasmuch as the subject is not unique to the structural discipline, the addressees were asked to pass copies of the questionnaire along to the other design section chiefs in their offices.

The great response to the questionnaire (82 replies) reinforces the writer's perception that there is a genuine concern for quality among the technical staff of the Corps of Engineers. It is sincerely desired that the work done here can result in a positive influence for our future.

Results of the Questionnaire

Before discussing the results of the questionnaire (Figure 1), it is appropriate to say a few words about the makeup of the respondents. The majority of the replies are from people who are considered the "backbone" of the Corps' design work. As Section Chiefs and senior technical experts, this group is responsible for assigning work to capable designers, applying sound engineering judgment, determining applicability of existing criteria and/or developing new ones, providing guidance and consultation, and assuring completeness and accuracy of design. As such, this group has the most direct and critical role in the success of the Corps'

[1] Chief, Design Branch, US Army Engineer District, Huntington; Huntington, WV.

1. How would you rate the overall quality of the Corps' products?
Poor_____ Average_29%_ Above Average_65%_ Excellent_6%

2. How do you see the trend for production of quality design in the future?
Declining_53%_ No Change_27%_ Improving_20%_

3. Do you feel there is an adequate balance between schedules and quality?
Too much emphasis on schedule_83%_
Adequate Balance_16%_
Too much emphasis on quality_1%_

4. Do you feel there has been a change in the emphasis on quality in recent years?
Less emphasis_54%_ No change_28%_ More emphasis_18%_

5. Does your office have a workable design quality management plan?
Yes_45%_ No_55%_

6. How does the upper management in your District promote and/or assure quality?
a. By including quality in the performance plan of employees._30%
b. By means of informal discussions with subordinates._28%
c. By enforcing a design quality management plan._19%
d. No action._23%

7. Do you feel sufficient resources are provided for accomplishing quality work?
Yes_40%_ No_60%_

8. For the design of Civil Works Projects, how would you rate the quality of the work by A-E's as compared to the Corps'?
Lower_71%_ Same_27%_ Higher_2%_

Figure 1. Design quality questionnaire (Sheet 1 of 3)

9. How would you rate the experience of the District's technical staff? Scale: 1(Poor) - 5(Highly)

	Average
a. Designers	3.82
b. Unit & Section Chiefs	3.95
c. Branch Chiefs	3.61
d. Division Chiefs	3.57

10. What is the attitude of the District staff toward quality?

	Unconcerned	Fully Committed
a. Designers	3%	97%
b. Unit & Section Chiefs	5%	95%
c. Branch & Division Chiefs	30%	70%
d. Others(Specify) LCPM	84%	16%

11. How would you rate the motivation of the District's technical staff? Scale: 1(Insufficient) - 3(Highly)

	Average
a. Designers	2.38
b. Supervisors	2.36
c. Branch & Division Chiefs	2.28

12. How would you rate the morale of the technical staff?
Scale: 1(Very low) - 4(Very high)

	Average
a. Designers	2.38
b. Unit & Section Chiefs	2.56
c. Branch & Division Chiefs	2.86

13. Do you feel the present grade structure of the District is adequate to retain experienced designers?
Inadequate 78% Adequate 22%

14. In the context of this subject, are there any constraints which impact your ability to produce quality work?

No 9%

Yes 91%: 1. Time and funding; 2. Lack of resources; 3. Too much resources devoted to "managing" projects; 4. Too many layers; 5. Lack of priorities; 6. Too much demand on technical staff to do administrative work.

Figure 1. (Sheet 2 of 3)

15. Is the organization of the Corps set up satisfactorily for producing quality design?

Yes 50%

No 50% : 1. Top heavy with PM's; 2. Lower grade of designers; 3. Too many administrative and support staff.

Number of people polled = 82

Discipline: (in descending order) Structural, Civil, General Engrg., Mechanical, Electrical, Architectural.

Position: (in descending order) Section Chief; Supervisory Design Engineer; Chief & Assistant Chief of Branch; Assistant Chief of Engineering.

Figure 1. (Sheet 3 of 3)

design mission, and their opinion should be valued.

After tabulating the responses to the questionnaire, it can be readily observed that a less promising trend exists, and it would be less than sincere to downplay the significance of the situation. The majority of the respondents feel that the Corps is producing above average quality products, which is an indication of their pride in the organization. However, over one-half of them say that there has been less emphasis on quality in recent years and that the trend for the production of quality design is declining. In answer to the question of balance between quality and schedules, an overwhelming 83 percent say there is too much emphasis on schedules at the expense of quality.

The survey shows that most offices do not have sufficient resources for accomplishing quality work and that the present grade structure of the Corps' Districts is inadequate for retaining experienced designers (questions 7 and 13, Figure 1).

When one considers the above answers and the others listed in Figure 1, the message coming across is very clear - we have been able to do a good job so far, but unless more emphasis is placed on quality, there will be problems. The Corps has not experienced a major problem to date, but the people who are responsible for its design mission are now raising the yellow flag. Considering the source, the importance of this message cannot be overemphasized.

Why Should the Corps Be Concerned about Quality?

Several years ago, it was determined that the Corps' projects from planning to construction took too long, and that accountability should be improved. A series of remedies aimed at improving the situation were put into motion by the then Assistant Secretary of the Army, Mr. Page. The crux of the changes, which everyone in the Corps is familiar with now, has to do with bringing the Corps more in line with the private sector. The impact of these changes will not be fully realized for a long time; however, the Corps is now more than ever conscious of the time and cost constraints.

Something that should have been considered was the differences between the nature of the Corps' business and that of the private sector. The Corps is the only nationwide water resources agency, and there are no engineering firms in the country with such a diverse, complex, and intricate mission as the Corps' Civil Works program. The limitations that make the Corps projects take so long have not and will not change due to several constraints such as local sponsor's funding capability, real estate requirements, Federal funding and budget cycle, endless involvement and coordination with local, state, and various federal agencies, environmental concerns, etc. None of these activities can be easily compressed, and design is the only area over which the Corps has complete control of scheduling and can shorten its duration - an activity that is not part of the problem in the first place. It is not difficult to predict the outcome. Without sufficient time and with the pressure to get the job done on schedule and within budget, quality will have to take the back seat.

Unlike private industry, there are no indicators in the Federal Government to warn the management that the "company" is in trouble because of the poor quality of its products. When the auto industry started producing shoddy products several years ago, the consumers finally got fed up and stopped buying their products. When the profits started going down, management got the message and started improving the quality of their cars and trucks, but not until they had lost a sizable share of the market to foreign car makers. In the case of Government agencies, how does upper management get the message about the quality of their products? After all, who is going to stop "buying" our products or return them under the "lemon law"?

In the service organizations of the Government, it may be understandable why there may be resistance to strive for quality and improved

efficiency. It takes an overall organizational commitment to achieve quality. This takes a tremendous amount of effort, dedication, and personal sacrifice. Considering the non-profit nature of the Government and the lack of meaningful incentives for its employees, it is easy to see why quality is only given lip service. While the lack of quality in service organizations results only in inefficiency and poor productivity, consequences of poor quality in an engineering organization will eventually lead to damages beyond anyone's imagination. This is because the only time it is realized that a serious problem exists in an engineering organization is after a major failure or catastrophe has occurred, e.g., the Teton Dam failure and the Challenger disaster. It is not suggested in any way that such situations exist today in the Corps; there are too many higher level reviews for a total failure condition to go unnoticed. But, as meeting schedules and budget become the central focus of the organization and with dwindling design and design review resources, it may be just a matter of time before something falls through the crack.

Quality Design in the Corps

Quality is a long-term investment. It does not happen accidentally without agreeing to pay for it up front and waiting for the results to start paying off. It is, therefore, a commitment that must start from the top. It is only the top management who can set the tone by declaring total organizational commitment to quality and allocate necessary resources. The commitment must be genuine and supported by real action, for engineers are too intelligent to be fooled by slogans, fads, or another regulation which requires reporting about quality. Once engineers are convinced that quality is top priority with management, then it becomes a philosophy which will be embraced by everyone in the organization.

The production of quality design requires adequate time, clear guidelines, and experienced designers. This is a simple process if there are no constraints. However, it is recognized that in a real-world situation there are both internal and external limitations within which the Corps is operating. The sizeable and complex organization of the Corps, its design, review and submittal process, and the intricate nature of its design mission place quality, schedules, and budget on divergent and, sometimes, conflicting paths. The challenge to the management is to eliminate or minimize those constraints over which the Corps has control. If that is to happen, the engineers of the Corps believe that the organization is fully capable of preserving the quality of its design while remaining sensitive to time and budget constraints.

Conclusion

Responding to the water resources needs of the nation and providing total engineering support for the Army demand the highest degree of professionalism, dedication, and state-of-the-art engineering. This is not an easy assignment. It requires ingenuity and, sometimes, trial and error to obtain the best solution. The engineers in the Corps work on some of the most difficult and complex projects, some of which have not been designed or built before. The Corps has been successful in accomplishing its mission so far. It appears that there is now a concern among the technical staff that the quality of the Corps' products in the future may be in jeopardy.

The questionnaire highlights several factors and a number of constraints which either directly or indirectly impact quality. The current perception is that while there are less resources available for technical work, the Corps is devoting increasingly more resources to managing, monitoring, and reporting the projects. The respondents also feel that there are too many layers in the organization and that there is too much demand on the technical staff to do more and more administrative work.

Fifty percent of the people polled believe the organization is top heavy with Project Managers, and they cite the low grade designers and the large number of support staff as detrimental to the Corps' mission.

A more detailed analysis of the situation and providing solutions for improvements or changes needed to assure quality in the Corps analysis and solutions are beyond the scope of this paper. However, the vital importance of the subject warrants a comprehensive study of design quality in the Corps. In order to obtain an unbiased evaluation, it is recommended that the study be performed by an ad hoc committee under authority of the Commander, USACE. To assure the objectivity of this committee, the members should be comprised primarily of engineers in grades 12 to 14 with direct design experience between 10 and 20 years. In view of the differences in the makeup of the Corps' Districts and Divisions, each office should be studied individually by means of onsite interviews with appropriate design personnel. The committee should be tasked to develop a report of its findings along with specific recommendations for improvements and/or changes needed. It is hoped that the results of this study will become the cornerstone of any future changes which may be occurring in the organization, and that they will contribute toward maintaining the Corps of Engineers' proud tradition of excellence.

Expedited Design and Review Procedures Used for Cuchillo Negro Dam, Truth or Consequences, NM

by

Raymond Veselka, PE[1]

Abstract

The project consists of a roller-compacted concrete flood control dam on Cuchillo Negro Creek to protect Truth or Consequences, NM. During preparation of plans and specifications, it became necessary to make major changes from the design approved in the Feature Design Memorandum because of cost overruns and foundation problems. This paper summarizes procedures used to expedite the design and approval process to complete the plans and specifications within the scheduled time and budget, while striving to obtain a quality design.

Introduction

Truth or Consequences, NM, is located along the Rio Grande in central New Mexico. Immediately upstream of Truth or Consequences is Elephant Butte Dam. Elephant Butte provides a flood control storage pool. There are two major tributaries to the Rio Grande below Elephant Butte Dam and above Truth or Consequences. These tributaries are the Cuchillo Negro Creek and Mescal Arroyo. Cuchillo Negro Creek poses a major flood threat to the Truth or Consequences area. The estimated discharge of the 1-percent chance flood from the Cuchillo Negro Creek is 50,000 cu ft/sec, and the maximum capacity of the Rio Grande through Truth or Consequences is 5,000 cu ft/sec. The plan of improvement consists of a dam on the Cuchillo Negro Creek to provide protection to Truth or Consequences from the 1-percent chance flood. Since all reservoir storage is for flood control, the dam reservoir is normally dry.

Project Design

Project design followed the usual Corps of Engineers (COE) procedure of Feasibility Report, General Design Memorandum (GDM) Number 1 which included detailed hydrology, and Feature Design Memorandum (FDM) Number 2 which presented design to serve as a basis for plans and specifications (P&S). The design consisted of a roller-compacted concrete (RCC) gravity dam section located in the Cuchillo Negro Creek canyon and a low earth-fill embankment at the left abutment. The RCC dam has a crest length of 550 ft and rises about 127 and 170 ft above the streambed and foundation grade, respectively. The outlet works, located at the left abutment of the RCC dam, consisted of an ungated two-way riser intake, a 7-ft-diam conduit, and a terminal energy dissipater. The spillway, designed to pass flows above the 1-percent chance flow up to and including the probable maximum flood, was located south of the main dam in

[1] Structural Engineer, US Army Engineer Division, Southwestern; Dallas, TX.

an unnamed tributary arroyo to the Cuchillo Negro Creek. The FDM RCC spillway consisted of a gravity section with a crest length of 1,100 ft rising about 60 and 90 ft above the streambed and foundation, respectively. A minimal outlet works with a 3-ft-diam conduit was provided through the base of the spillway. Another dam feature was an 800-ft-wide channel cut/borrow source in the ridge separating the Cuchillo Negro canyon from the adjacent canyon with the spillway. A project site plan is shown in Figure 1, and FDM spillway and dam cross sections are shown in Figures 2 and 3.

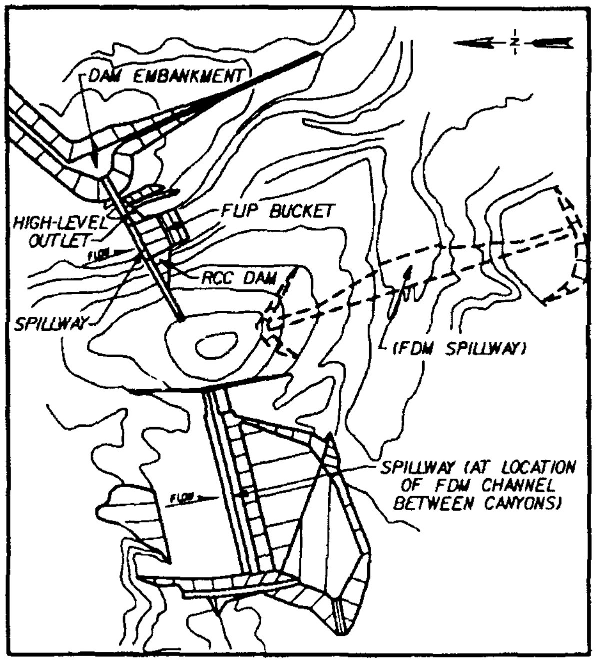


Figure 1. Site Plan

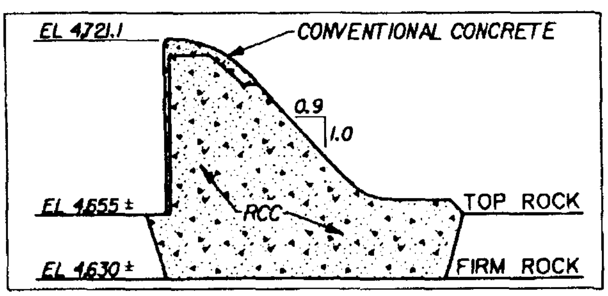


Figure 2. FDM spillway section

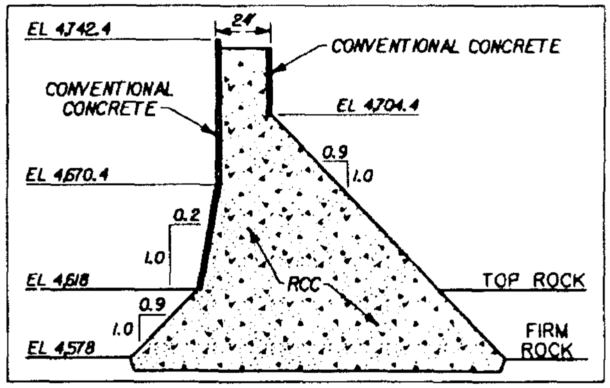


Figure 3. FDM dam section

Expedited Design and Review

FDM 2 was approved in November 1988, subject to comments. Comments included requirements for further subsurface investigations and testing to evaluate the foundation adequacy. (Note that COE regulations define the design contained in an FDM to be the final design with only the completion of details to be performed during preparation of P&S). Boyle Engineering Corporation, Albuquerque, NM (AE) was directed to proceed with preparation of P&S while the COE continued with foundation explorations. Also to expedite work, it was agreed that review and approval of design would be accomplished through onboard reviews, as needed, followed by an onboard review of final P&S. The management decision to use these expedited design and review procedures was made to meet the scheduled FY 88 new start, September 1989 construction award date.

Events overtook this management plan. The additional foundation exploration indicated that a substantial increase in excavation would be necessary at the spillway and raised questions about the adequacy of the spillway foundation. Project costs increased because of the increase in foundation excavation and exceeded the $16 million cost limit in the Local Cooperation Agreement. The local sponsor did not have funds to cover an increase in the project cost.

The first onboard review on 26 and 27 April 1989 was expanded to include CECW,

CESWD, CESWA, and the AE, with Managers, Hydraulic, Civil, Structural, and Soils Engineers, and Geologists from each office, a board of foundation consultants, and an RCC expert from Portland District. There were contentious discussions about the adequacy of the spillway foundation, but this issue was sidestepped. Because of the cost overrun for the project, CESWA proposed to move the spillway to another site. The spillway would be relocated to the site where the channel cut through the ridge between the Cuchillo Negro Canyon and adjacent canyon in the original FDM 2. Foundation exploration at the new site was limited, but foundation exploration available indicated that an ogee weir with an RCC chute spillway was feasible. The width of the spillway weir needed to be as short as possible because the dip of the bedrock would cause the chute cost to increase rapidly as the weir width increased. This led to a decision to add an ogee weir spillway through a notch in the crest of the main RCC dam. There were hydraulic discussions on acceptable flow depths over the spillways and energy dispassion and discussions on structural design of the ogee weir and RCC chute. This meeting resulted in management decisions to move and redesign the spillway, perform additional foundation explorations, and to to meet the September 1989 construction award date.

The AE and CESWA presented the revised design at a second full- staff onboard review meeting on 20 and 21 June 1989. Major dam features included a 650-ft-wide chute spillway cut into the ridge between the two canyons with an ogee weir crest at elevation 4721.1 and a 3-ft-thick RCC chute slab with a slope of 1 vertical on 3 horizontal. The overdam spillway is 120 ft wide with an ogee weir crest at elevation 4721.1, variable height training walls on the downstream face of the RCC dam, and a flip bucket at the toe of the dam. The outlet includes a high-level, ungated outlet box, 7 ft, 2 in. high by 10 ft wide, through the RCC dam at el 4,674 ft located at the contact of the RCC and left abutment, and a 5-ft-diam ungated outlet near the base of the dam.

Foundation exploration for the chute spillway showed a limestone near the streambed of the canyon overlain by a conglomerate. Information about the conglomerate did not conclusively show if this strata was adequate to prevent erosion at toe of the chute or to clearly establish its elevation at the west side of the chute. The P&S were with the chute terminating at the limestone. The chute slab was to be constructed in 1-ft lifts laid down in 10-ft-wide lanes with upper lanes stepped up the slope to form a slab about 3-1/4 ft thick. There were discussions about weathering and potential for erosion of the RCC chute slab. It was decided to increase the lane paving width to 12 ft to thicken the chute slab to allow for erosion and weathering. There were discussions on analysis of stability for the spillway ogee and chute, on the need for a subdrain system below the chute, and on potential cracking of the RCC chute. A subdrain system consisting of a 1-ft-thick filter below the chute slab and rows of weep holes at a vertical spacing of 10 ft was provided. A supplement to the FDM was needed to document the revised design, and the supplement should be submitted as soon as possible after completion of P&S. Figure 4 shows a profile of the canyon and dam looking downstream and Figures 5 and 6 show cross sections through the revised spillway and RCC dam.

The advance final on board review was held on 11 and 12 July 1989. This review resulted in normal review comments mainly about details. P&S were advertised on 1 August 1989.

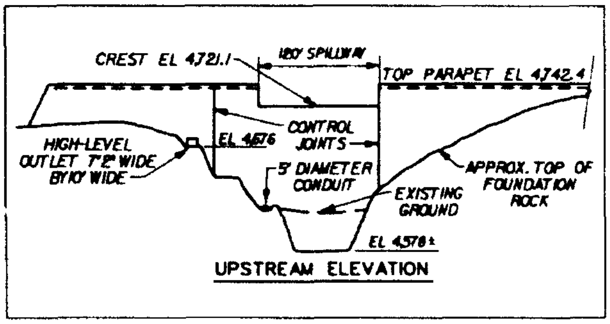


Figure 4. Upstream elevation RCC dam

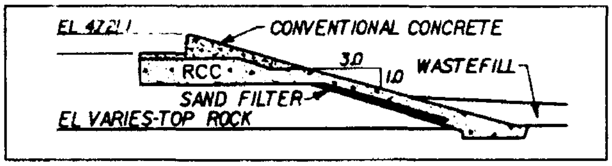


Figure 5. P&S spillway section

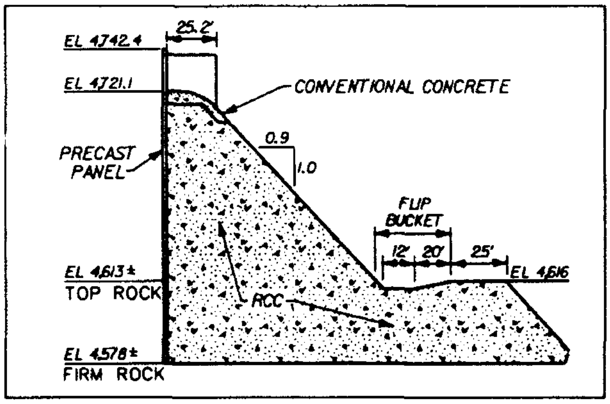


Figure 6. P&S dam section

Design Changes During Construction

After excavation for the spillway chute reached the conglomerate, this strata was adequate to prevent erosion at the bottom of the chute. The construction contract was modified to terminate the chute at this strata. This modification was for a credit of $450,000.

During preparation of the dam foundation, questionable rock was found in the left abutment. It should be noted that abutment foundation exploration that should have been performed for the FDM and also as a result of FDM comments was not performed prior to construction. Initially, CESWA Construction and Geotechinacal Engineers made decisions about foundation changes. There was not sufficient coordination about how the foundation conditions and revisions were affecting the other aspects of the dam design. Design issues included dam sliding stability, foundation differential settlement, foundation leakage, potential for dam cracking, and erosion of the abutment caused by discharge from the high-level box outfall. Eventually Hydraulic Engineers, Structural Engineers, the AE, foundation consultants, CESWD, and CECW became involved. The following revisions were made to the dam as the result of the foundation conditions. Additional foundation exploration was performed at an estimated cost of about $18,000. The intake of the high-level outlet was modified and a U-frame flume was added downstream of the outlet through the RCC dam. Cost for this modification was $57,736. Additional left abutment excavation, rock bolting, and dental concrete foundation preparation was performed at a cost of about $1.4 million. Two control joints were added through the RCC dam with a modification cost of $229,00C. Costs for additional dam facing panels, additional RCC, and additional erosion protection downstream of the high-level box outlet U-frame flume are not available at the time this report is being prepared. Figure 7 shows details of the foundation at the dam left abutment.

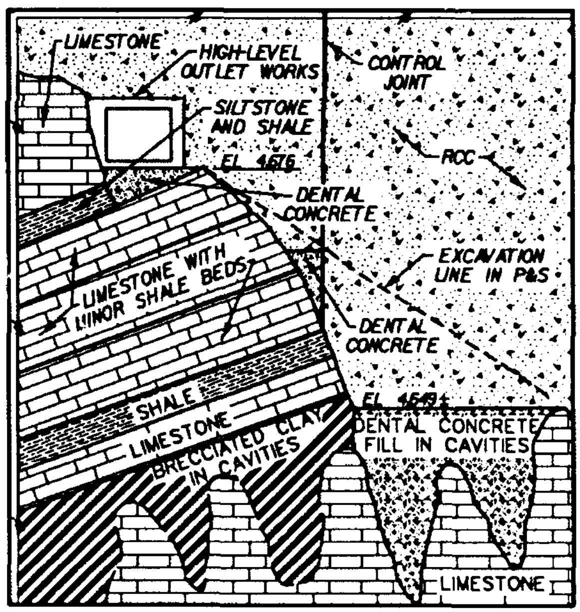


Figure 7. Left dam abutment foundation

Summary

Extensive redesign, review, and approval for the dam was accomplished in a short 3-1/2 months. The design was driven by the management emphasis on meeting schedules and

containing costs even when mountainous unforeseen design obstacles appeared. There was considerable lost design effort because of the redesign. Fortunately, the AE designer was experienced and cooperative. The COE wisely used the expertise of managers and a great number of engineering disciplines from many different offices. Design changes became necessary during construction due to unknown foundation conditions. If the abutment conditions had been known before completion of P&S, the design solution would have been different. The cost changes resulting from the redesign during construction are a credit of $450,000, an increase of about $1.7 million. The Contractor is a tough negotiator, and contract modifications cost substantially more than the cost would have been if the changes had been made prior to advertising the P&S. Contract modification costs plus other Contractor claims will increase the final dam cost to about $17 million.

Conclusions

There is a real risk that a quality design will not be achieved when expedited design and review procedures are used. Expedited design requires the use of experienced designers that know design criteria and have highly developed engineering judgment. Fast design and onboard reviews may not allow sufficient time to address or understand all critical design elements as adequately as the proven method of preparing and reviewing DM supplements to present revised design. With fast design and review there is an increased risk that the structure may not completely fulfill its design purposes or that there will be future costs to correct design deficiencies. The ultimate risk could be structural failure.

Professionalism places a responsibility on technical designers to continually evaluate the design as it evolves to ensure that a quality design is achieved. At any time when design problems or schedule and money constraints raise doubts about the ability to complete a quality design, the designers must forcefully communicate the problem to management. Technical designers must also emphasize to managers and COE construction personnel of the need for a proactive, not reactive, engineering role during construction. Management and COE construction personnel must learn to be more sensitive to the concerns of technical designers in the interest of building a quality project.

Olmsted Locks and Dam

by

Holly A. Gittings, PE,[1] *and Jeffrey E. Bayers, EIT*[2]

Abstract

A General Design Memorandum (GDM) on the Olmsted Locks and Dam Project was submitted in March 1989. The project featured in the GDM consisted of two locks, a spillway section controlled by tainter gates, a navigable pass operated by wickets, and a fixed weir. Subsequent to completion of the GDM, four dam configurations were evaluated and compared on several factors to determine the best alternative. The plan that was eventually selected consists of two locks, a navigable pass/regulatory section controlled by wickets, and a fixed weir. A hydraulic wicket was chosen for operation of the navigable pass/regulatory section. The above as well as other modifications to the GDM recommendation were presented in a GDM Supplement, which was submitted in April 1990. Studies are now underway in preparation of the Feature Design Memorandum (FDM) on the locks. The locks are a W-Frame structure with a dual culvert system in the middle wall. The structure will be supported by a pile foundation. The program CWFRAM, which was developed from the Corps library program CUFRAM, has been used to design the structure for static and pseudo-static conditions. The dynamic analysis will be performed by a design response spectrum procedure using the general purpose finite element programs ADINA and GTSTRUDL. A version of CWFRAM that incorporates dynamic capabilities is presently under development and will also be used. In the preliminary design of the pile foundation, a rigid base analysis using the program CPGA for static and pseudo-static conditions was performed in combination with a flexible base analysis using CWFRAM. A dynamic analysis of the pile foundation using the program CPGD will be performed in future work. Either ADINA or GTSTRUDL will additionally be used. In conjunction with the design of the locks, a nonlinear, incremental structural analysis is being performed.

The Lock FDM is currently scheduled to be submitted in 1991. Other studies that are planned include the preparation of plans and specifications for the navigable pass prototype gate and the investigation of alternate methods for constructing the dam to save time and cost.

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Introduction

The Olmsted Locks and Dam Project will replace existing Locks and Dams 52 and 53 and will be located at Ohio River Mile 964.4, approximately 1.8 miles downstream of Locks and Dam 53. Locks and Dam 52 is located approximately 20 miles downstream of Smithland Locks and Dam, a project that was completed in 1980. Locks and Dam 53 is located approximately 24 miles downstream of Locks and Dam 52 and 18 miles above the mouth of the Ohio River.

The original Locks and Dam 52 project was completed in 1928. It consisted of a single 110-foot wide by 600-foot long lock; a 1,958-foot wide, manually (boat) operated wicket dam; three beartraps for pool control; and a fixed weir section. An austere 110-foot wide by 1,200-foot long lock was completed during 1969 as a temporary measure to reduce excessive traffic delays. The sheet pile structure was never intended for long-term use. The filling and emptying system is considerably less efficient than those of the modernized locks on the Ohio River.

Locks and Dam 53 is very similar to 52. The original project, completed in 1929, consisted of a 110-foot wide by 600-foot long lock; a 1,880-foot wide, manually operated wicket dam; two beartraps; and a fixed weir. A temporary 110-foot wide by 1,200-foot long lock was completed in 1980.

The wicket dams of these projects can be lowered when river flows are sufficient to provide the required navigable depth. This creates a navigable pass for towboats and eliminates lockages. This open river condition exists at Locks and Dam 52 about 60 percent of the time and at Locks and Dam 53 about 90 percent of the time.

The Olmsted Project will provide a navigation pool to the Smithland Locks and Dam (River Mile 918.5). Subsequent to completion of the new project, Locks and Dams 52 and 53 will be dismantled as necessary to allow safe navigation through the project area.

Design Memorandum No. 1, General Design Memorandum (GDM), for the Replacement of Locks and Dams 52 and 53, Olmsted Site, was submitted for review on 30 March 1989 and approved on 26 September 1989. The Olmsted Locks and Dam, as presented in the GDM, consisted of two 110-foot wide by 1,200-foot long locks adjacent to the Illinois bank, a spillway section controlled by six 110-foot wide tainter gates, a 1,125-foot wide navigable pass comprised of wickets, and a 650-foot wide fixed weir extending to the Kentucky bank.

Throughout the development of the Olmsted Project, there has been much discussion about the design characteristics of the dam. Initial emphasis, during the Lower Ohio River Navigation Feasibility Study and early GDM phases, was placed on the navigable pass gate design. As the project design progressed, interest developed in alternatives for eliminating the tainter gate piers partly due to seismic design requirements. These considerations led to the identification and assessment of four alternative dam configuration plans. Model studies of the four dam configurations were performed by the Waterways Experiment Station (WES) on the 1:120 scale general navigation physical model. Detailed quantity take-off and unit cost values were developed for the design and construction cost estimate of each alternative plan. Major Repair cost estimates were also prepared for the plans. Annual operation and maintenance costs were estimated based upon the Louisville District's experience with locks and dams on the Ohio River. In the examination of alternative plans, several non-economic factors or qualitative factors considered to be important in assessing the four plans were identified. These factors were operability, maintainability, navigability, constructibility, environmental, earthquake resistance, and safety. After a thorough and extensive evaluation of the four alternative dam configurations, an all wicket dam was selected as the preferred plan for future design studies. This plan, shown in Figure 1, consists of two 110-foot wide by 1,200-foot long locks, a 2,200-foot wide navigable pass/regulatory section controlled by 220 ten-foot wide

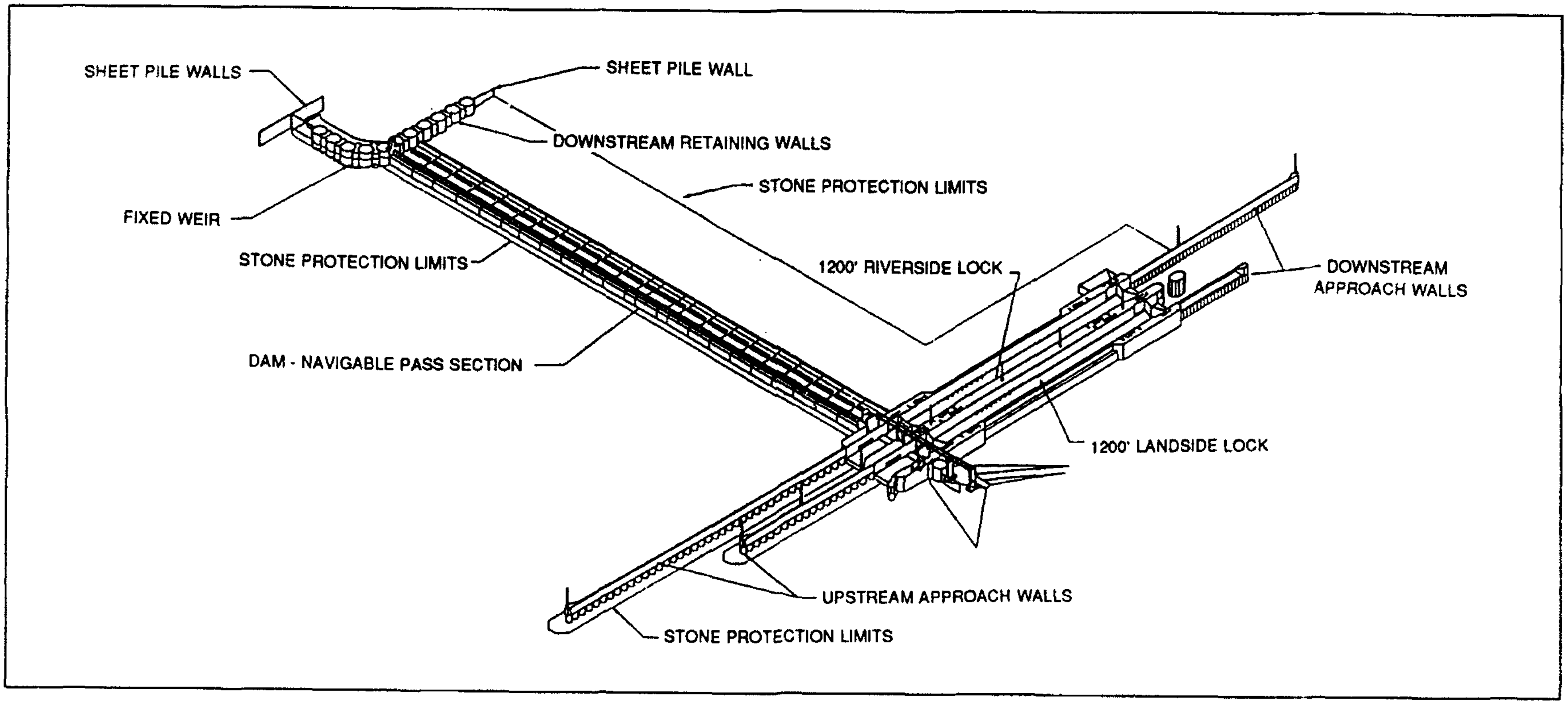

Figure 1. Pictorial view of project

hydraulically operated wickets, and a 426-foot wide fixed weir.

Modifications to the design of Olmsted Locks and Dam resulting from comments on the GDM and from changes in the dam configuration and project scope as presented in the GDM were featured in a GDM Supplement. Supplement No. 1 to Design Memorandum No. 1 was submitted for review on 30 April 1990 and approved on 22 August 1990. The following paragraphs discuss the major project features contained in the GDM Supplement.

Locks

The 110-foot wide by 1,200-foot long lock chambers are formed by nineteen successive W-Frame structure monoliths supported by a pile foundation. Figure 2 shows a 3-D view of a miter gate monolith developed on an Intergraph workstation using Intergraph Engineering Modeling System. The middle lock wall provides a dual filling and emptying culvert system with a minimum width. The culverts are higher and narrower in the upper miter gate monolith than in the rest of the middle wall. The length of the W-Frame portion of the locks is minimized by locating the discharge bucket as far upstream as practical. Because independent culverts are provided for each lock, a model study of the filling and emptying system will not be required. However, the discharge bucket, with four outflow culverts, will require a small hydraulic model. A high-level bulkhead/access bridge spans over the locks and to the Illinois bank. The emergency bulkheads for the locks will be stored under this bridge in the extended area between the land lock and the bank.

Much work has been accomplished in the design of the lock structure and foundation, and more analysis with more sophisticated tools is planned to be done and incorporated in the final design. To date, stability studies and analyses of static load conditions have been completed for typical cross sections along the length of the lock. Pseudo-static earthquake analyses on a typical lock chamber cross section have been completed, and dynamic analyses are

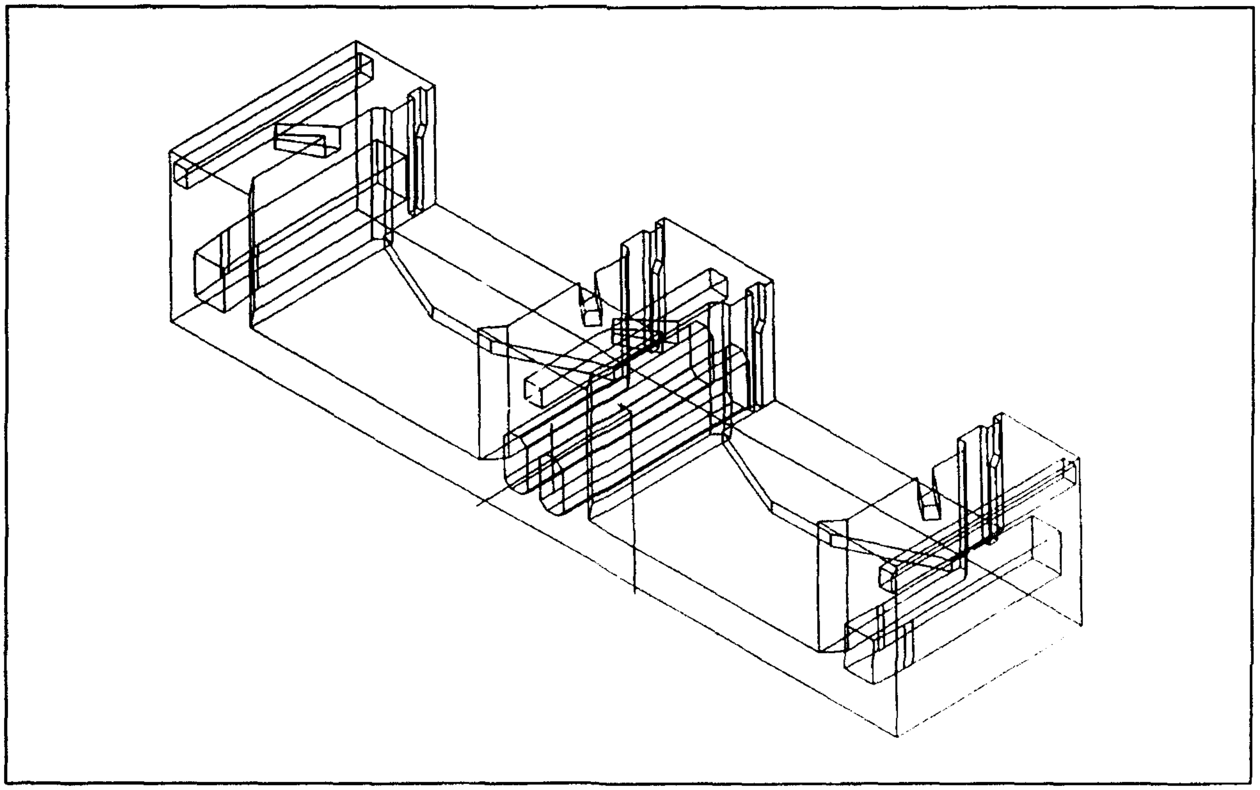

Figure 2. Miter gate monolith

underway at present. In addition, a nonlinear, incremental structural analysis is also being performed. The combination of analysis techniques and tools will allow for parametric considerations and comparisons that should yield a comprehensive, economical design in which an excellent understanding of structural performance can be achieved.

The major design tools in use at this time consist of various software programs with different levels of capability, types of results, and hardware requirements. The majority of design work accomplished thus far has centered around the use of the Corps' Computer Aided Structural Engineering (CASE) Task Group programs CPGA and CWFRAM. The CASE Pile Group Analysis (CPGA) program allows for a rigid base analysis of the pile foundation in three dimensions. Pile forces, pile head displacements, and pile bending and axial interaction values are obtained and used for design purposes. However, a rigid base analysis gives linear distribution of axial load across the base, and with the size of the base and high concentrations of axial load under the walls of the W-Frame cross section, questions arise on the validity of this analysis. The CASE program CWFRAM, a two-dimensional analysis of U-Frame or W-Frame structures, was developed for the Olmsted Project from the original CUFRAM program. This analysis incorporates a flexible base analysis and is more accurate in computing the pile forces since it redistributes the forces with respect to the flexibility in the base of the structure. In addition to pile forces and displacements, it also gives axial forces, shears, and moments for the superstructure members. This provides a means of designing the reinforcement in the structure, and gives values for comparison to the results of the more sophisticated finite element analyses that are currently being performed. Comparing the values from the two programs shows that, in fact, a flexible base analysis is warranted. While the pile loads out of CPGA are nearly equal across the section, the CWFRAM runs show significant redistribution of the forces from the centers of the chambers to the areas under the walls. Due to this study, approximately 15 percent of the piling has been eliminated out of the middle of the chambers without significant change in loading on the piles under the walls. Of course, the lateral loads increase when fewer piles are present, but these loads served as the limiting factor for exactly how many piles to remove out of the cross section. The only drawback to the CWFRAM program is the absence of a 3-D capability, but, due to the relatively consistent cross-sectional geometry along the length of the lock structure, the drawback is minimized. The CPGA program allows 3-D analysis and used in combination with the CWFRAM can give good indications of 3-D effects. The earthquake loading conditions in both of these programs are in the form of pseudo-static conditions. A version of CPGA called CPGD does allow for a response spectrum analysis of earthquake forces and will be used in future refinements of the design. At the present time, development of a dynamic capability within the CWFRAM program is underway. When this is complete, it too will be utilized in further development of the understanding of the dynamic response of the W-Frame structure.

Preliminary dynamic analyses were performed on the lock structure using the GTSTRUDL finite element analysis program and a response spectrum procedure. Supplementary dynamic analysis work is being performed at WES. This dynamic analysis work is utilizing a response spectrum analysis using the finite element program ADINA on the new CRAY computer at WES. Additionally, new response spectra are being developed and will be incorporated in the work at WES. Also, a time history analysis will be performed to gain a deeper understanding of the dynamic response of the structure, and a dynamic analysis of the pile and soil foundation interaction using the SUPERFLUSH FEM Program is being considered.

In addition to the dynamic analyses that are taking place and that are planned for the future, a nonlinear, incremental structural analysis (NISA) study is also currently underway. This study will provide detailed information on the thermal stresses associated with the construction methods and sequencing

during construction phases due to the nonlinearity of the concrete material properties during curing. Preliminary results of this study, which is being performed at WES using the ABAQUS FEM Program, have yielded very positive results. These results have indicated that thermal stresses seen with the proposed construction sequence and methods do not approach allowable limits and actually moderate further after a 240-day period. The concrete mixes studied and the results achieved show that a pozzolan replacement with fly ash of 40 to 50 percent is possible without detrimental effect. Also, the study indicates that a 120-day strength mix will be suitable. These results are preliminary, but confidence in them at this point is high.

The studies completed to date and the future planned effort will be integrated into the final design of the lock structure for a comprehensive understanding of the performance of the structure throughout its economic life. This parametric approach should yield a high degree of confidence in the total design quality of the project.

Approach Walls

The approach walls facilitate alignment of vessels entering and exiting the lock chambers. The design for both the upper and lower approach walls consists of a concrete monolith placed on circular steel sheet pile cells filled with stone. H-piling will be driven inside each cell. Revisions to this design, including the deletion of the internal H-piling and the use of precast concrete beams to eliminate the cast-in-place concrete that falls below normal construction period water levels, are being investigated.

Navigable Pass Gates

The 2,200-foot wide wicket dam will serve as both a navigable pass and a regulatory section for the hinged pool. The wickets will be raised and lowered as required to hold the upper pool as low as possible while maintaining elevation 300 at Paducah, Kentucky. The upper pool will not be drawn down below elevation 295. The upper pool elevation will be maintained by the wickets with these objectives in mind and, during the transition period from locking to open river, will also be operated to minimize project swellhead so as to effect a smooth merger between the pools. The navigable pass will not be available for vessel transit until the tailwater has risen to elevation 295. There will be occasional short periods when the tailwater will rise above elevation 295, but, due to low flows on the Ohio River, the navigable pass cannot be dropped because the proper pool elevation cannot be maintained at Paducah. The navigable pass will be open to navigation approximately 58 percent of the time, although this could vary from year to year.

There will be a total of 220 hydraulic wickets, each having a 10-foot nominal width with 4 inches between them and a length of 26 feet for the 2,200-foot navigable pass dam. The wickets will be hinged at the base and raised to an angle of 65 degrees to the horizontal, leaning in the downstream direction. A 10-foot by 10-foot dry gallery will span the navigable pass under the wickets. Hydraulic cylinders located under the wickets in a wet chamber will raise the wickets from the lowered horizontal position of -3.83 degrees to the raised angle of 65 degrees. A prop and hurter will be used to hold the wicket in the raised position similar to the existing wicket design used at Locks and Dams 52 and 53. The controlling load condition is raising the wicket with the wicket in the lowered position, upper pool at elevation 300, and lower pool at elevation 284. The top of wicket elevation will be 300.5 with the wicket raised.

To raise a wicket, the piston rod of a 15-inch bore hydraulic cylinder located vertically under the lowered wicket is extended, as shown in Figure 3. The wicket raises, and the cylinder rotates through an arc as the wicket reaches its raised position of 65 degrees. The prop is attached to the wicket and follows a guided path in the hurter. A signal will be sent to the control station once the piston rod has extended far enough to allow the prop to be set in a notch in the hurter. Once this position is reached, the

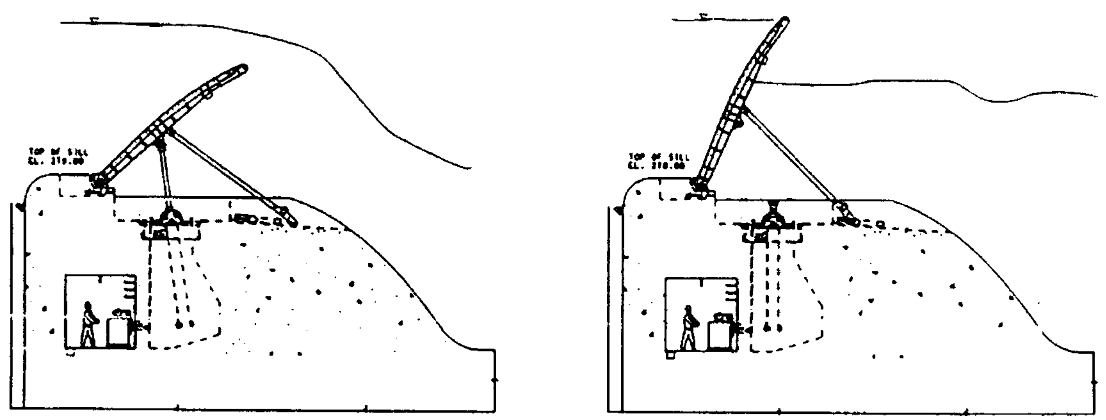

Figure 3. Wicket in raised position

piston rod is retracted back into the cylinder and out of the river environment. Figure 3 shows the wicket in the raised position. To trip the prop and lower a raised wicket, the piston rod is again extended up to the wicket and releases the prop. To align the 15-inch bore cylinder with the contact point on the wicket, a second smaller cylinder, which is attached to the side of the 15-inch cylinder, is first extended its full stroke. Once this piston rod is extended, the larger cylinder will be in proper alignment. The piston rod raises the wicket from its position of 65 degrees to 70 degrees, as shown in Figure 3. This releases the prop from the notch in the hurter and allows the wicket to be lowered back to the sill by the cylinder. Figure 4 shows the wicket in the lowered position. The entire process to raise one wicket requires approximately 15 minutes. Retraction of the piston rod back into the cylinder also requires about 15 minutes. In lowering a raised wicket, it is expected to take approximately 15 minutes to trip the prop and 5 minutes to allow the piston and wicket to lower back to the sill.

Operation of the wickets will be from the control structure located on the middle lock wall. The wickets are also designed to be raised and lowered under flowing conditions by a crane located on a work boat if the hydraulic system is inoperable.

It is planned to construct and test a full-scale prototype at the Smithland Locks and Dam facility. The prototype will consist of five wickets and their associated components and machinery. It will be located in a channel that is excavated around the fixed weir. Bulkheads will be placed upstream and downstream of the wickets to allow dewatering during testing for inspection purposes. The prototype will be constructed with several different components and various materials, which will enable designers to refine components and to select the best materials for use in future construction of the Olmsted Dam. In addition, the stilling basin will be constructed to aid in the developing of dispersing flow. After construction of the dam, the prototype will be used for testing any necessary changes in the wicket design before actual installation at the Olmsted site.

Cofferdams and Construction Sequence

Conventional cellular steel sheet pile cofferdams will be used to construct the locks and dam. The construction will be accomplished in four stages. Acceptable navigation conditions will be maintained at all times.

The locks and the connection to the Illinois bank will be constructed in the first stage

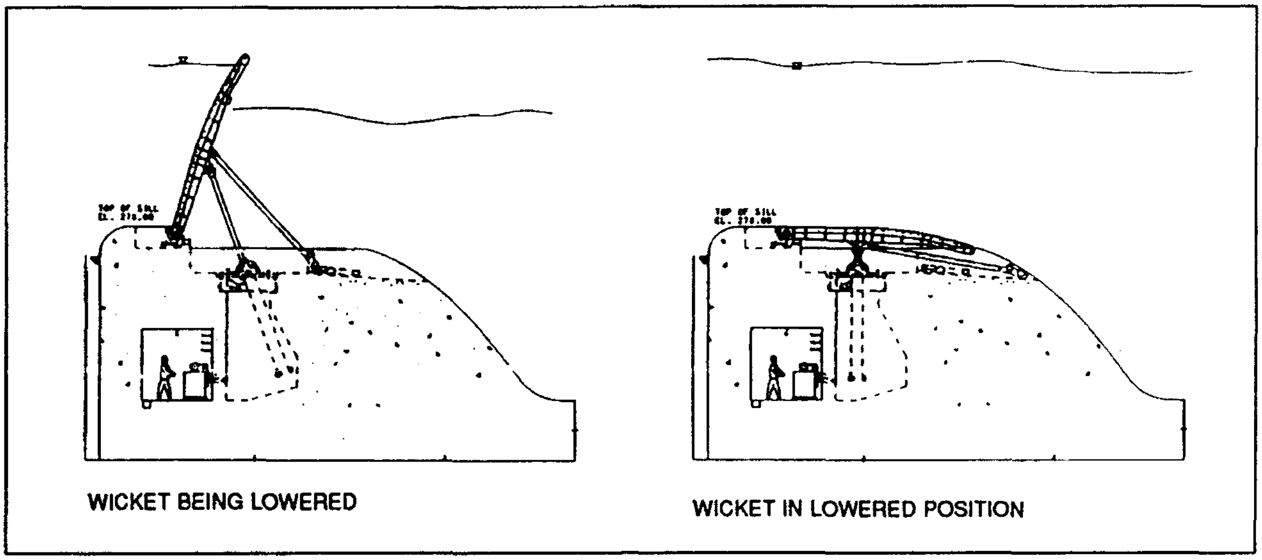

Figure 4. Wicket in lowered position

cofferdam. This cofferdam should have little effect on river traffic. The dam will be constructed in the remaining three cofferdam stages. An approximate 1,100 feet of navigable pass will be constructed in the second stage cofferdam, which will tie in to the river lock wall. The locks will not be in service during this stage of construction. All river traffic must pass around the cofferdam. A bypass channel will be excavated on the Kentucky side of the river to accommodate traffic during periods of low flow. The locks will be available for river traffic during low flow periods in the third stage of construction. During periods of high flow, river traffic will travel over the previously completed section of the navigable pass. An approximate 700 feet of pass will be constructed in this stage. In the fourth stage of construction, river navigation will utilize the same traffic pattern as described for the third stage. The fixed weir and the rest of the navigable pass, about 400 feet, will be constructed in this stage.

Use of the conventional cofferdam is the only method of construction considered to be applicable for the locks. However, alternative construction methods that would save time and cost are under consideration for the navigable pass portion of the dam.

Conclusion

Feature Design Memoranda and plans and specifications are currently being prepared for various project features. These documents include the Lock Feature Design Memorandum and the navigable pass prototype gate plans and specifications. The first construction contract for Olmsted Locks and Dam is scheduled to be awarded in 1992.

Portugues Dam Monolith Layout and Survey Control

by
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Abstract

Portugues Dam will be the first three-centered, double curvature, concrete arch dam to be designed and constructed by the US Army Corps of Engineers. Its unique arch shape provides substantial savings in concrete over conventional gravity dams. Accurate placement of concrete monolith formwork is essential to produce the arch geometry which matches the design layout. Careful planning and execution is required before and during construction to ensure that appropriate tolerances are met. This paper describes the planning, methods, and considerations involved in developing the survey control and the procedures for locating excavation limits and positioning of the concrete formwork for the mass concrete placement.

Design Quality During Survey Plan Formulation

Structural involvement during the design of Portugues Dam included identification of site location, iterative layouts and preliminary analyses to arrive at a final dam design, static, dynamic, and thermal stress analyses to verify a satisfactory design, and preparation of design memoranda and contract drawings to present the final design to higher authority review and prospective contractors. Even after the final shape of the structure was defined and approved, structural involvement has not ended there. US Army Engineer District, Jacksonville, structural engineers are working closely with survey and construction personnel to ensure the constructability of the finished product within acceptable standards of accuracy for concrete placement. This involvement and coordination with other disciplines is one way that design quality is being achieved on Portugues Dam.

Background

The Portugues Dam site is located in the southwest corner of the island of Puerto Rico, just north of the city of Ponce. The Portugues and Cerrillos Dams, along with debris basins, drop structures, and channel sections, comprise the Portugues and Bucana Rivers Project which provides flood protection to the city of Ponce. Construction of the Cerrillos Dam (embankment) has just been completed. Construction of the Portugues Dam will begin in early 1992 and will be divided into four contracts as shown in Table 1.

Construction of the dam structure will be split between two contracts, the interim dam construction and the water supply addition. The interim dam provides flood protection only with a crest elevation at 534.6 ft. Increasing the height of the structure to a crest elevation of 585.6 ft to include reservoir capacity for water supply will be funded

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100 percent by local sponsor costs and will be scheduled when funds become available. At the streambed, the elevation at the base of the dam is 315 ft which translates into a structural height of 219.6 ft for the interim dam and 270.6 ft for the multipurpose dam (flood control and water supply). The crest length around the axis of the dam is 1,316.781 ft for the interim dam and 1,505.143 ft for the multi-purpose dam.

Table 1
Portugues Dam Construction Schedule

Contract	Scheduled Start Date	Scheduled Duration, months
Foundation Excavation	Mar 1992	12
Foundation Grouting	Oct 1994	12
Interim Dam Construction	May 1996	24
Water Supply Addition	To be scheduled	15

Portugues Dam is designed as a double-curvature, three-centered arch dam. The term "double-curvature" refers to the fact that the upstream and downstream faces are defined by both vertical **and** horizontal curvature. The plan view shown in Figure 1 reveals the horizontal curvature. The vertical curvature can be seen in Figure 2 where a section has been taken through the dam at the point of maximum height (crown cantilever). It can be seen in Figure 3 that the horizontal curvature of the arch shape is a combination of three tangential arcs (two outer segments and one inner segment) each with its own respective center of curvature. This defines the shape as being "three-centered."

Because Portugues Dam is a mass concrete structure, nonlinear incremental stress analyses were performed to determine thermal effects from the concrete heat of hydration, solar radiation, and reservoir temperature. The results of these analyses were used to determine the concrete monolith width and lift height to be specified for concrete placement. For Portugues dam, a monolith width of 70 ft and a lift height of 10 ft will be used.

Planning of Construction Control Surveys

Arch dams provide substantial cost savings in concrete over conventional gravity dams. Properly designed, the arch shape results in a structure in which most of the elements are in compression. This allows the use of a thinner section by taking advantage of the compressive strength of concrete. At the same time, construction is complicated somewhat due to the forming of curved upstream and downstream faces. Accurate procedures and quality control during construction operations is necessary to construct a structure which accurately reflects the shape modeled during design/analysis.

During the planning for the construction control surveys and methods, literature on the subject of arch dam construction was researched. Most of the arch dam construction in the United States has been performed by the Bureau of Reclamation. The methods used for the construction of Morrow Point Dam in 1968 (Willis 1972) were reviewed for general and site specific considerations for possible application at Portugues Dam.

Because of the critical need to place formwork accurately (in general, more accurate than is required on most other types of Corps structures), it was decided early on that a first-order network of monuments to be used during construction would have to be established. Two issues regarding this decision immediately arose. The first concerned standard Corps surveying practice. For the most part, with some exceptions, the Corps of Engineers uses third-order surveying procedures. At the district level, experience with first-order construction surveying methods is limited. Discussions between the US Army Engineer District, Jacksonville, surveying personnel and surveyors at the Engineering Technology Laboratory and Headquarters, US Army Corps of Engineers, were necessary to formulate plans for the establishment of a first-order network at the Portugues Dam site. The second issue concerned the absence of any first-order control existing on the island of Puerto Rico. An

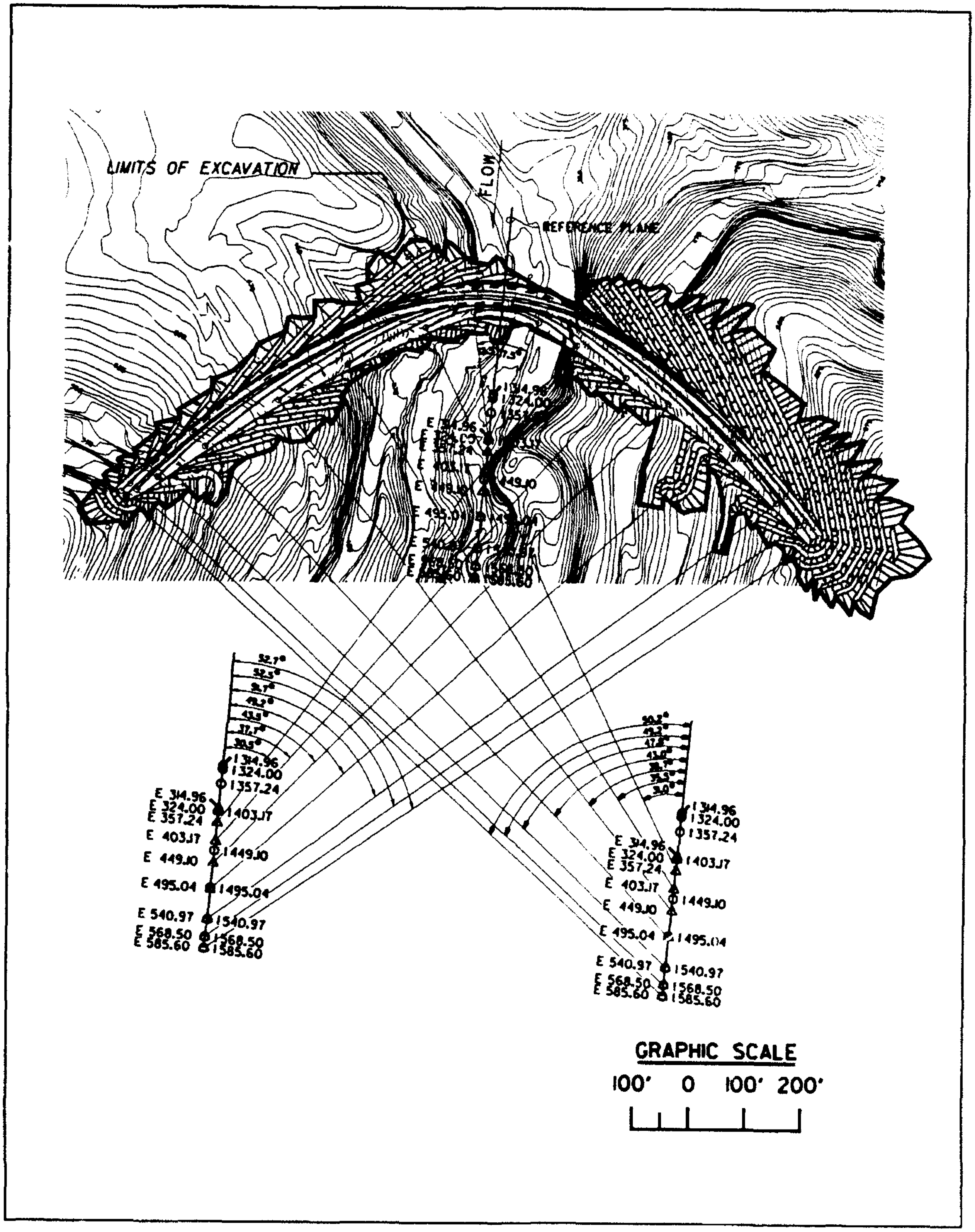

Figure 1. Plan view of Portugues Dam

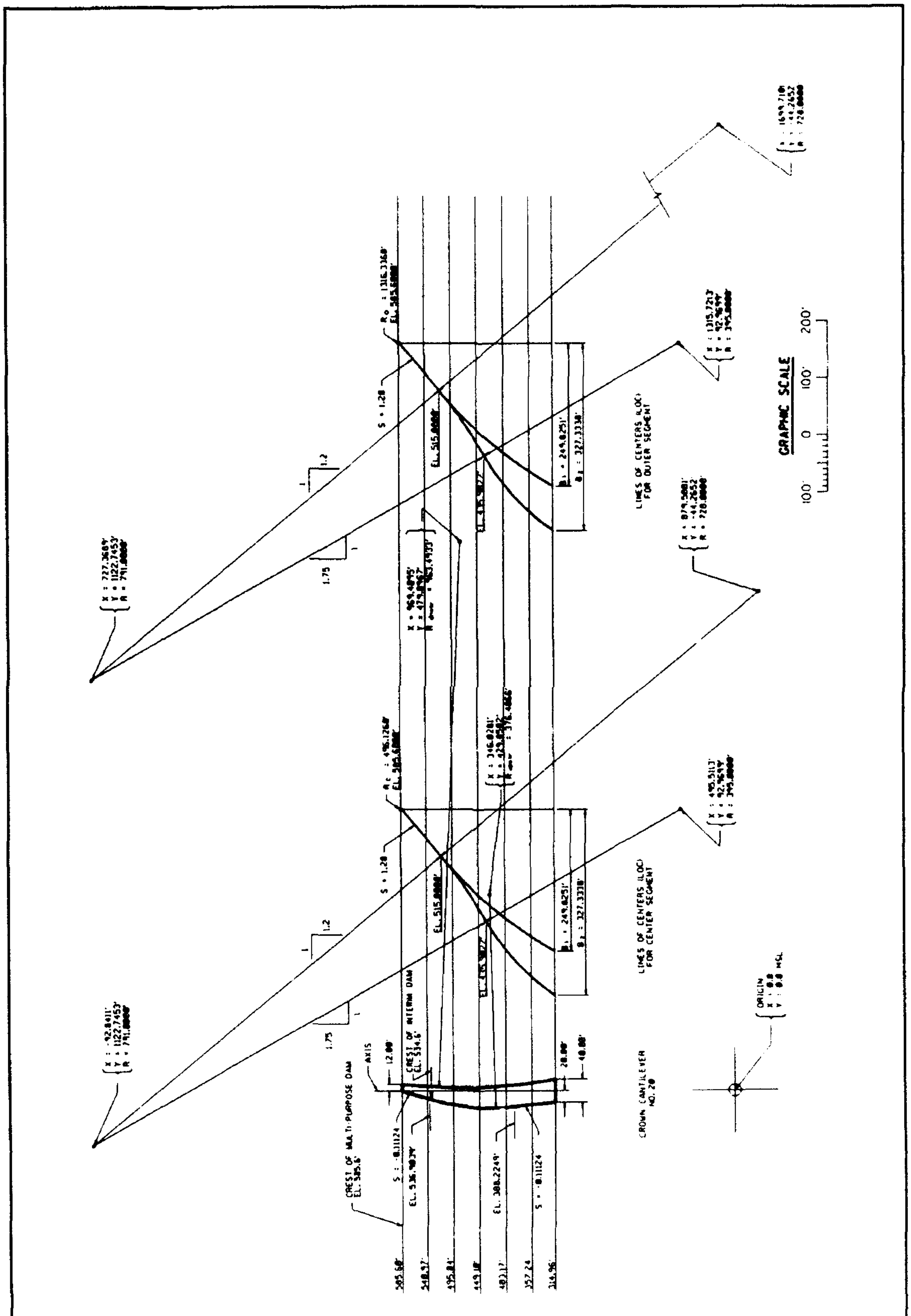

Figure 2. Section along reference plane

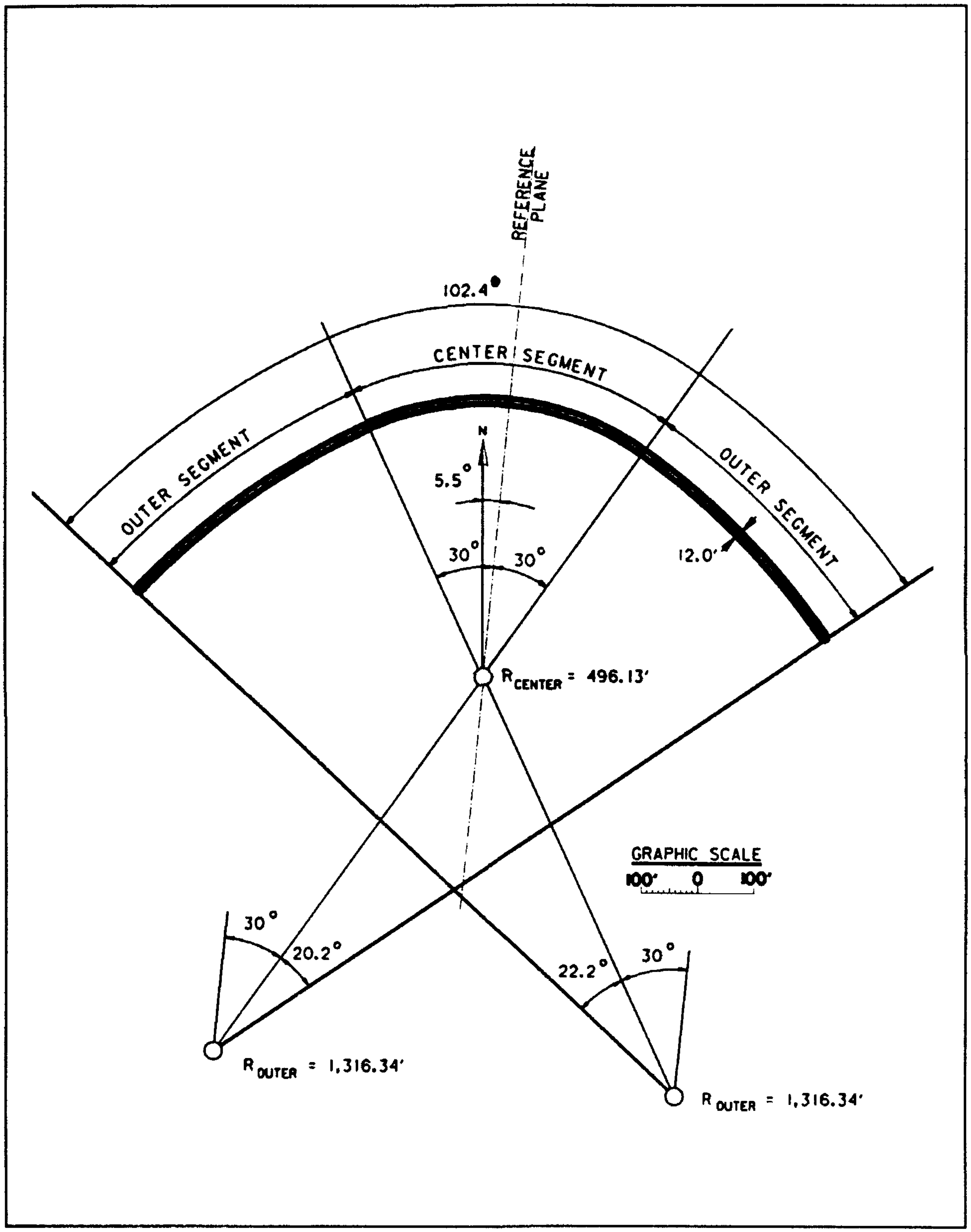

Figure 3. Arch section at crest (spillway not shown)

examination of existing monuments at the damsite revealed two second-order monuments in the vicinity of the dam. However, the absence of any first-order control on the island proved to be a minor concern, as will be discussed later.

An ad-hoc committee, composed of members from Structures Section, Survey Branch, Geotechnical Branch, Real Estate Division, and Construction Operations Quality Assurance, was formed to review the construction control survey plan and to ensure all inter-disciplinary considerations were addressed prior to contract advertisement.

Detailed Considerations of the Construction Control Surveys

Figures 1 and 2 contain all the geometric data required to define the shape of the arch dam at any location on the upstream or downstream face on the dam body. A computer program (ADCOP) was written which inputs all this geometry data in addition to the station values of desired vertical construction joints (based on the results of the thermal stress analysis, as previously discussed) and the elevations of concrete lifts. ADCOP computes the coordinates, based on a user-defined coordinate system, at the upstream and downstream face of the intersection of each lift line and vertical construction joint or, in simpler terms, the corners of each monolith block. However, because of the length (70 ft) of the monoliths, intermediate points at a closer spacing are required to produce an acceptable amount of horizontal curvature. These intermediate points are obtained by adding station values to the input data file. The coordinates of these points will be presented in the contract drawings, and the contractor will be required to locate his formwork so that it passes through these specified points. Figure 4 shows a typical section with associated coordinate information that will be presented in the contract drawings.

In all, 22 lifts of concrete will be placed during the flood-control dam contract with 5 additional lifts placed during the water supply addition contract. The 22 concrete monolith sections in the contract drawings contain a large number of formwork coordinates that will be relayed to the contractor. Because of this quantity of tabulated coordinates, it was advantageous to develop a local project datum coordinate system for the excavation and dam contracts rather than using the state plane coordinate system. The reasons for this were twofold. First, the state plane coordinate system is not oriented with the reference plane of the dam, a major feature on which most of the dam geometry is defined. Secondly, several revisions to the state plane coordinate system exist (e.g. 1927 datum, 1983 datum) which could provide an element of confusion to the contractor. Establishing a local project datum with one axis parallel to the reference plane of the dam provides a clearer and more logical system to both the designer and to the contractor. Figure 5 shows the local project datum.

A first-order network of construction monuments is required to accurately locate grade limits for the excavation contract and concrete block formwork for the interim dam and water supply addition contracts. As mentioned earlier, no first-order control exists on the island of Puerto Rico. Establishing a first-order network based on the state plane coordinate system would have been difficult, if not impossible. The decision to use a local project datum negates this problem, however. Criteria followed when selecting locations for the construction control monuments were as followed:

- Locations to be between (approximately) 500 and 1,000 ft from dam.
- Each monument to provide visibility to **most** areas of dam/excavation.
- Total network to provide visibility to **all** areas of dam/excavation (no blind spots).
- Monuments to be located in areas easy to access and where possibility of disturbance from contractors operations is unlikely.

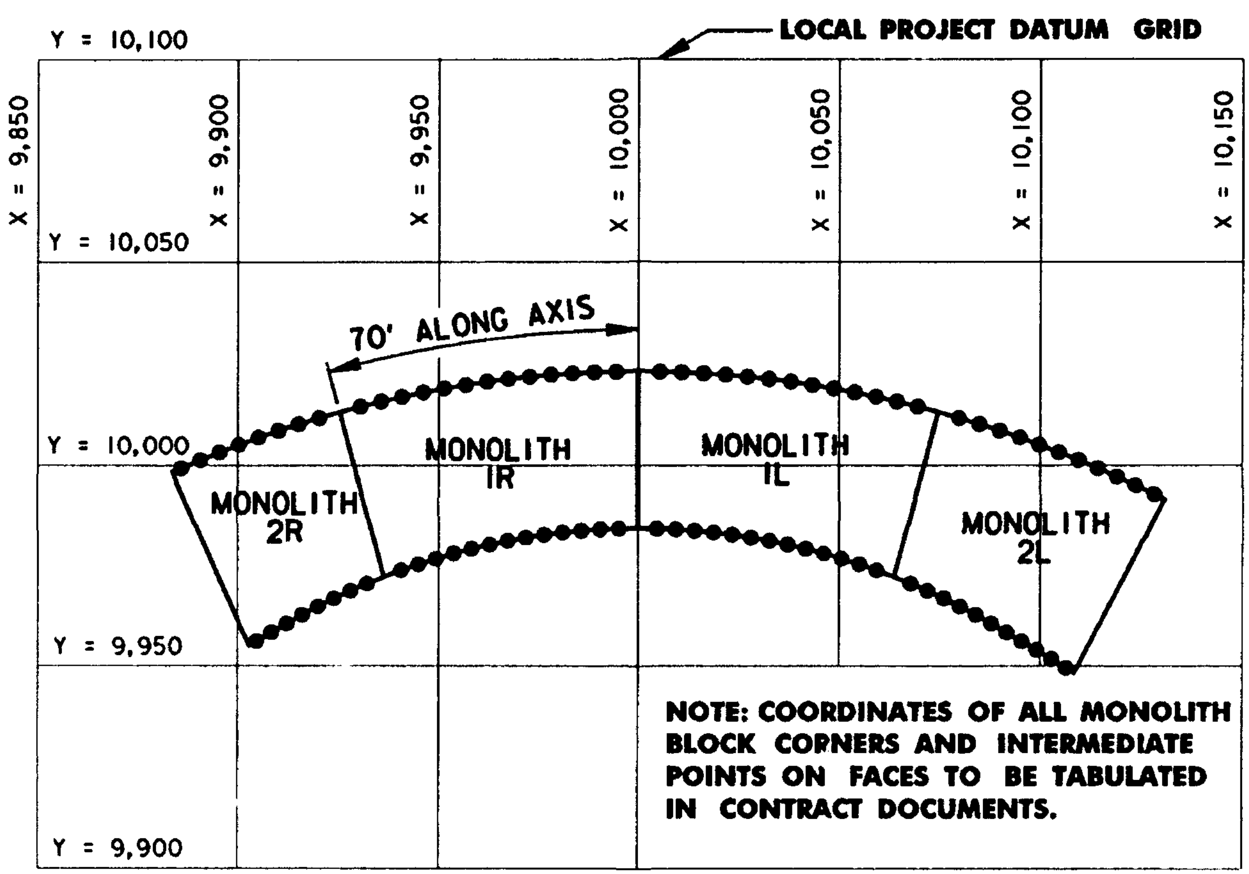

Figure 4. Typical arch section

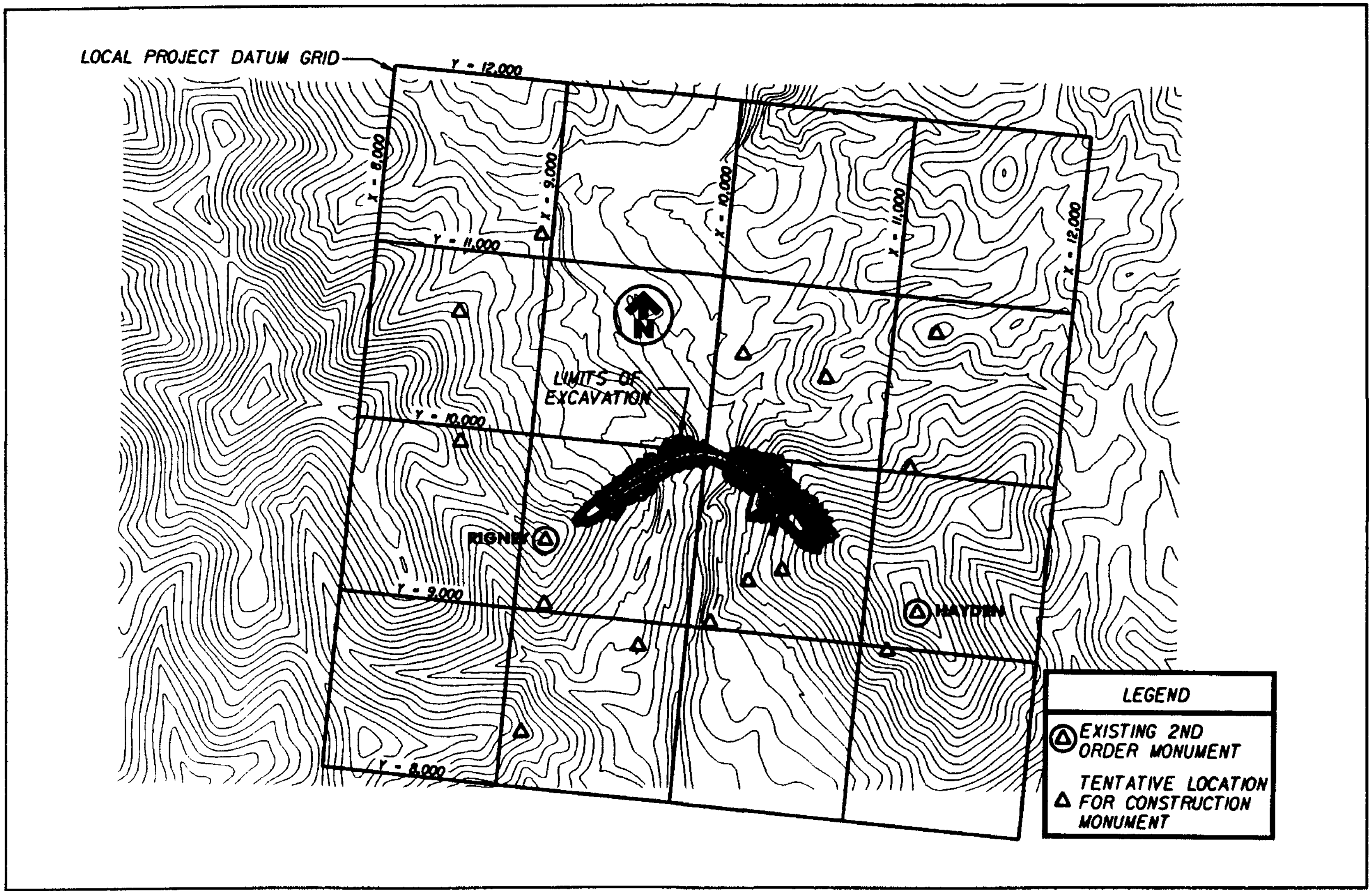

Figure 5. Site plan showing construction control monument locations

The network was to be established prior to advertisement of the excavation contract to include those sheets into the contract drawings. Figure 5 shows tentative locations for the construction network monuments. Final locations are pending a site visit, scheduled for mid-June 1991.

Positioning of Monolith Block Formwork

Coordinates of the construction monument network will be referenced to the local project datum in the contract drawings. The contractor will begin placing formwork for each block by first selecting a pair of monuments that he prefers to occupy during the positioning process. The first monument will be used to locate the formwork; the second will be used by the contractor to verify its final position (Figure 6). The formwork coordinates from the contract drawings along with the monument coordinates provide the contractor with an azimuth and a distance to shoot from his selected monuments. As a minimum, the contractor will be required to use a total station surveying instrument with a least count of 2 sec. After locating all block corners and intermediate points from the first monument, the contractor will be required to verify the final formwork position from another (separate) monument. Once this has been completed, a Corps A-E surveyor will perform a quality assurance check from two separate monuments, other than the monuments the contractor has occupied, to verify the formwork position prior to the placement of any concrete.

Ongoing Studies

While many details concerning the monolith block have been finalized, some items involved are still being investigated.

As mentioned before, coordinates of intermediate points along the upstream and downstream face will be supplied in the contract drawings in addition to the block corners to produce an acceptable amount of curvature in the finished concrete face. The spacing to be used for these intermediate points is under investigation. Spacing used at Morrow Point dam (Willis 1972) was 6.0 ft for all blocks except at Block 10, near the center, where a spacing of 4.5 ft was used. Because Morrow Point Dam was a single-centered arch dam, arch radii (horizontal) throughout the dam were nearly uniform and a constant spacing value (6.0 ft) could be used throughout most of the structure. In contrast, Portugues Dam, with its three-centered geometry, has much shorter horizontal radii in the inner segment than the radii in the outer segment. Spacing of intermediate points based on the degree of curvature may be different (and closer) in the inner segment than the spacing used in the outer segment. Reducing the intermediate point spacing in areas of the dam body where the degree of curvature allows may be specified to simplify formwork erection which may result in lower construction costs.

In addition to the formwork erected to form the upstream and downstream faces of the dam structure, internal formwork and blockouts will be required to form galleries, plumblines, and other features. Forming of these internal recesses may be more complicated than forming the external face, in some cases. Two options for locating the internal formwork are being considered. The first is to specify the location of formwork by local project datum coordinates; the second is to use offsets from block corners or some other point within the monolith.

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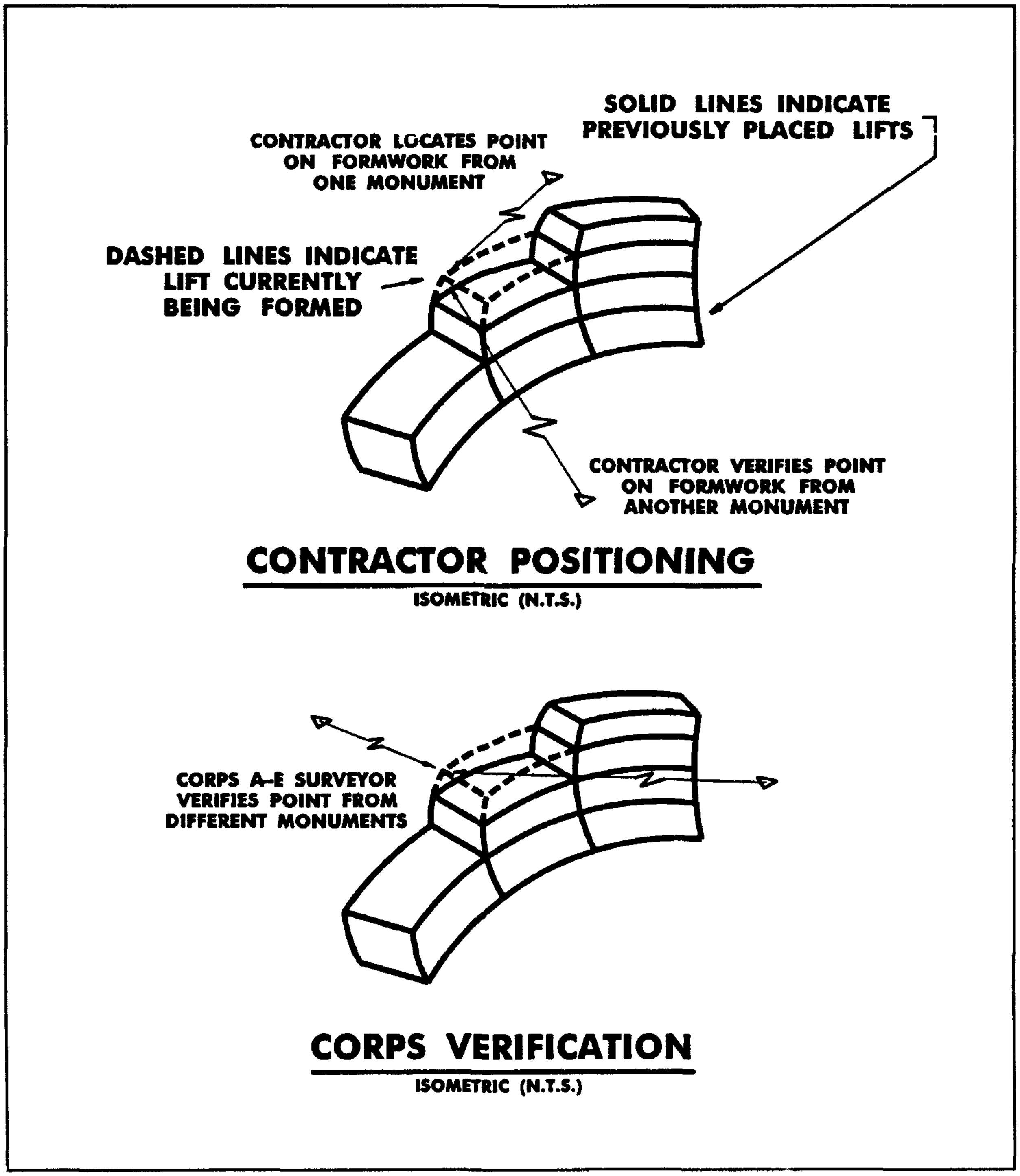

Figure 6. Construction layout and QA verification procedures

Cofferdam Design Problems
Point Marion Lock, Monongahela River, Pennsylvania

by
John C. Gribar, PE[1]

Abstract

Point Marion Lock and Dam is located on the Monongahela River, south of Pittsburgh, Pennsylvania, at the West Virginia border. The construction of the new lock is located adjacent to and landward of the existing lock. The back of the new river wall is located approximately 10 ft from the back of the existing land wall, and its foundation is 8 ft lower. The use of the existing land wall as the main arm of the cofferdam with underlying weak foundation material created the need for extensive rock anchor work and very restrictive installation and excavation procedures. Extensive design analyses were performed, and some innovative design approaches are being used for construction.

Introduction

Point Marion Lock and Dam is situated on the Monongahela River, 90.8 miles above Pittsburgh, Pennsylvania. This is actually south of Pittsburgh at the Pennsylvania-West Virginia border as the Monongahela River flows north. The existing lock is located on the left bank and consists of a single lock chamber, 56 ft wide by 360 ft long. It was constructed by Government forces in 1923-1926 and placed in service in October 1925. The dam was constructed during the same period and originally consisted of a 560-ft-long fixed-crest structure. It was modernized in 1958-1959 to provide a gated structure and raise the pool by 4 ft. The dam was rehabilitated in 1987-1988 by using rock anchors to meet current stability criteria and performing concrete repairs and painting.

The new lock is located landward of and adjacent to the existing lock and has the same top of wall elevation 803.0. The lock will be a single chamber, 84 ft wide by 720 ft long, and will have the downstream pintle on a line with the downstream pintle of the existing lock. The back of the new river wall will generally be 10 ft from the back of the existing land wall, but will be as close as 8 ft away. The meandering configuration of the river in this area, as well as its location with respect to the Cheat River and a major power plant upstream, prevented us from locating the lock at another site. There will not be any changes in the upstream pool, elevation 797.0, and the downstream pool, elevation 778.0. Upon completion of the new lock, the existing lock will be removed and a fixed weir section will be constructed between the new lock and the existing dam.

Foundations

Foundations for the existing lock structure and the new lock are primarily fine-grained

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sedimentary rocks, such as claystones, weak indurated clays, and siltstones overlying a 2- to 3-ft-thick coal seam. The strength parameters of these materials are extremely low, and some core borings revealed very poor contact zones between the concrete and the indurated clay. A stability analysis of the existing land wall showed a failure to meet present-day criteria for both normal operating and extreme maintenance conditions. Below the coal and underlying indurated clay is a unit of siltstone that grades to a hard sandstone. Construction of a new lock chamber only 10 ft landward of the existing lock presents special problems with regard to the river wall foundation. The ideal foundation would be in the siltstone below the coal, which is as deep as 35 ft below the existing land wall. Excavation to this depth would create an extremely difficult and expensive problem to maintain the stability of the existing lock land wall, which will remain in service. Boats will continue to use the lock during construction of the new lock. The decision was made to raise the foundation to weaker, but firm rock, 8 to 12 ft below the existing land wall foundation, which is generally at elevation 760 (+/-).

The foundations of the new walls will be at elevation 752 or 749 in rock units consisting of primarily interbedded claystone and siltstone. The new lock walls were dimensionally designed to meet criteria for overturning, and the weak rock units were compensated for in-sliding by incorporating a 3-ft-thick anchored and reinforced concrete chamber floor which would function as a strut between the two walls. This strut would accommodate sliding tendencies induced by differential pool conditions during normal operation and maintenance conditions.

Main Cofferdam Arm

Because the locks are adjacent to one another, the existing land wall is being utilized as the main arm of the cofferdam. The relatively small size of the sections and the poor foundation condition prevented the landwall from meeting stability criteria for the cofferdam condition. Stressed rock anchors (Figure 1) were installed vertically at the river face of the land wall to resist overturning. A total of 139 vertical rock anchors were installed in the 32 existing land wall monoliths to provide overturning stability. All anchors were stressed to a working load of 422 kips before excavation behind the land wall could begin. An inclined row of stressed rock anchors were installed at about one-third the height of the monolith, approximate elevation 771.0, on the land side to resist sliding forces. Excavation was generally limited to elevation 780.0 until these 157 anchors were installed and stressed to a working load of 442 kips. This was the lowest elevation at which these anchors could be installed to obtain a temporary factor of safety of 1.25. Once two anchors were stressed in a monolith, excavation could proceed to elevation 766.0. These anchors had to be carefully located to be between the vertical anchors and also at a position to miss the bottom corner of the culvert in consideration of drilling tolerances. A row of toe blocks with stressed inclined anchors (a total of 129 anchors) was then installed at the bottom of the land wall sections to provide adequate factors of safety for deep-seated failures into the adjacent excavation for the new river wall. This excavation would be 8 to 12 ft below the foundation of the existing wall and have its face 8 to 10 ft away. At 16 monoliths, the rock is not strong enough to carry the bearing pressures of the thrust blocks. In order to transfer the load from the anchor to a more competent rock, a caisson (Figure 2) with thrust block was designed. A total of 74 such caisson-thrust blocks will be used. A total of 55 conventional toe block anchors are also being used. Again, these three rows of anchors had to be carefully located to avoid interference with one another and engage a sufficient volume of rock at the anchorage length to meet the factors of safety. The existing monolith joints were drilled to provide a 6-in.-diam hole centered on the joint and grouted to prevent leakage into the dewatered area.

In order to install the upper inclined anchors in the land wall and lower guide wall, special restrictions and installation procedures had to be developed. With the need to

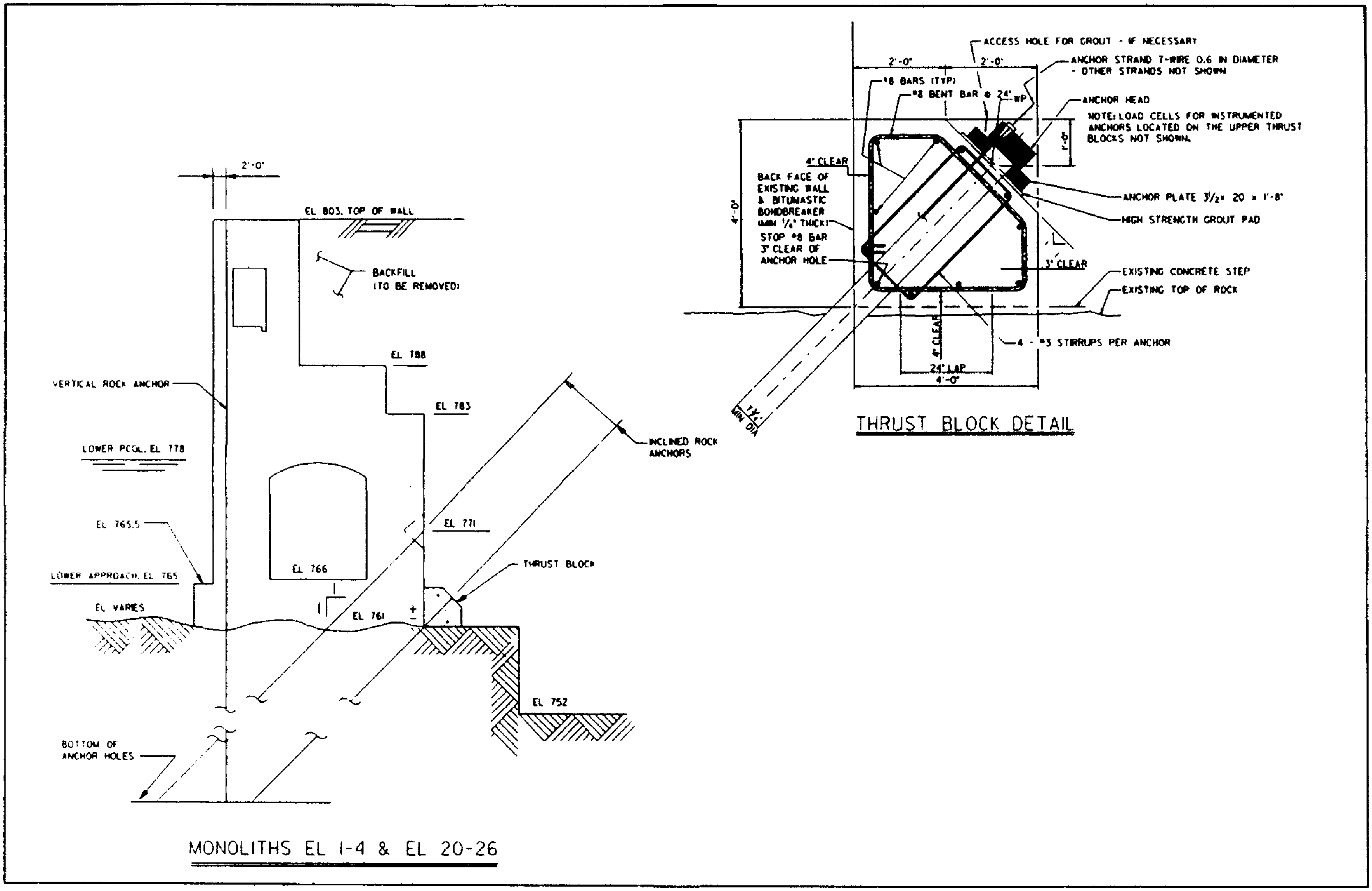

Figure 1. Point Marion Lock cofferdam existing land wall thrust blocks

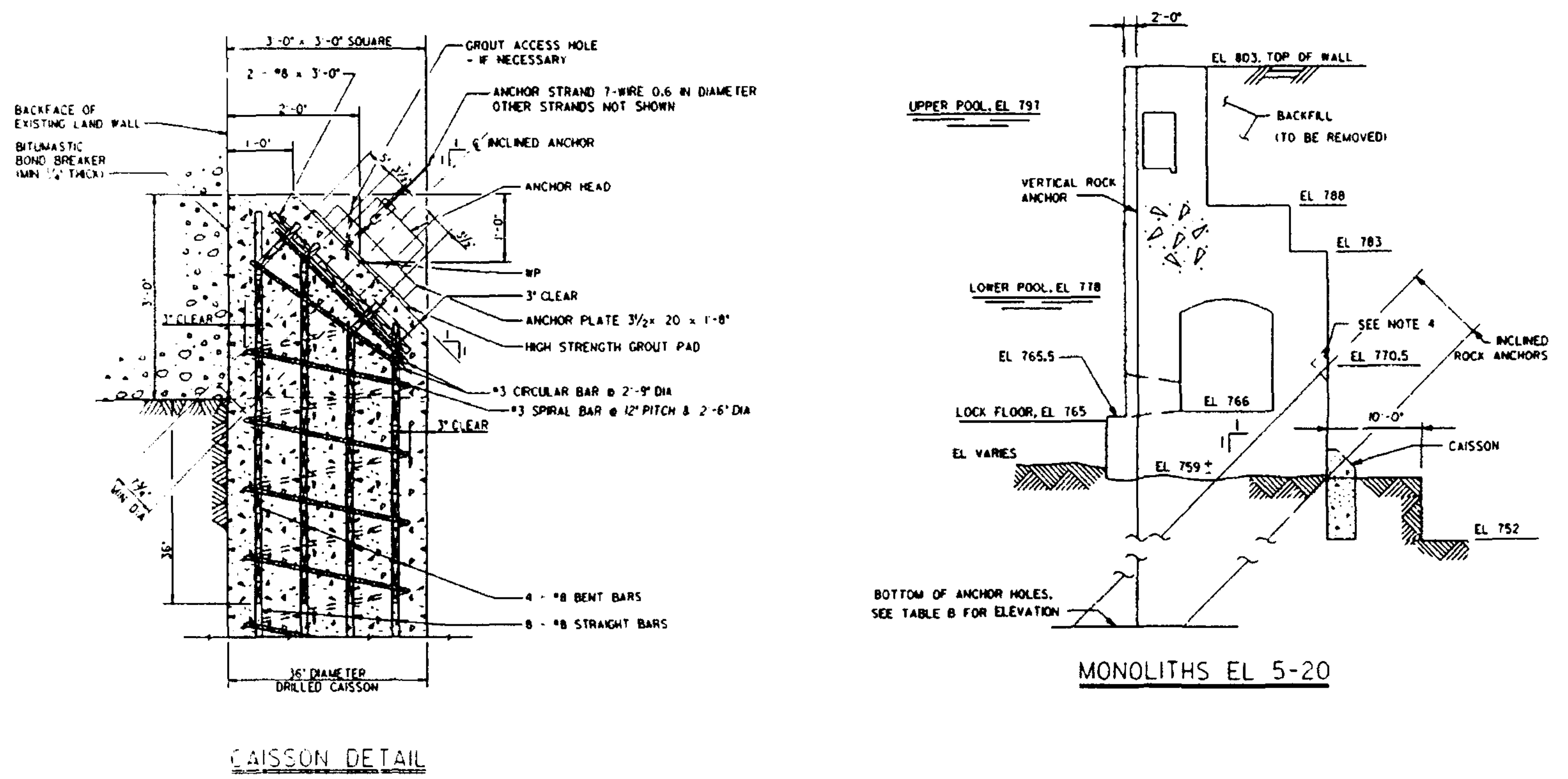

Figure 2. Point Marion Lock cofferdam existing land wall caissons

install the anchors at a 1-on-1 slope to provide the necessary restraining forces, there was a problem in installing the anchor past the lower corner of the culvert while maintaining the height of the backfill to achieve the necessary factor of safety. A steel-fabricated stressing frame (Figure 3) was devised to permit installation of these anchors. This framework had an angled guide for accurate drilling and aligning of the hole and was hung from the step above at the back of the monolith. A temporary excavation was made locally at the back of the wall, and the stressing frame was installed. The excavation was backfilled to elevation 774.0 and shaped to prescribed limits. The anchor intersected the back of the wall at elevation 770.5. This allowed the anchor to be installed from a point 5 ft higher than without the frame and maintained the required factor of safety of 1.25 as a temporary condition. Excavation was limited to one stressing frame at a time per monolith.

Lower Cofferdam Closure

Traditional 50-ft-diam cells seated into rock with interconnecting arcs and filled with pervious material are provided at the down stream end of the lower guide wall. Five cells were constructed extending downstream approximately 300 ft, and two cells were extended at a skew landward to close the cofferdam into the bank. The top of the cofferdam is at elevation 800, which corresponds to an average annual recurrence interval of 12 years. The cells were capped with 2 ft of riprap, and the primary floodway was provided in this section of the cofferdam. The new river wall ends at about halfway down the existing lower guide wall and posed no restrictions for the lower cofferdam. Twenty-six toe blocks with 14-strand stressed anchors were constructed at the base of the cells to prevent sliding.

Upstream Cofferdam Closure

The closure of the cofferdam at the upstream end posed special problems due to space restrictions. The new river wall extended upstream beyond the existing guide wall, and the two walls were 10 ft apart, back-to-back. There was also a need to maintain uninterrupted service through the existing lock, and thus the need to maintain the line of river face of the land wall extended upstream. After evaluating several alternatives, it was decided to use relatively small, 25-ft-diam concrete-filled cells, which are not independently stable, in this restricted area. These cells extended approximately 150 ft upstream from the guide wall and are provided with stressed rock anchors (Figure 4) to meet required factors of safety. The concrete fill is necessary to withstand the substantial anchor forces without cell deformation or settlement. All anchors are stressed to a working load of 492 kips. Fifty-foot-diameter free-standing cells, seated into rock and filled with pervious material, then turn landward into the bank to form the upstream closure.

The concrete-filled cells have slightly inclined top-stressed anchors to resist overturning forces and upper inclined anchors at the inside face of the cells to resist sliding forces. These upper inclined anchors are located at elevation 773.0, an elevation that would provide a factor of safety of 1.25, the minimum allowable. These anchors could not be provided at a lower elevation or at the base of the cells, as the temporary factor of safety during installation would be less than 1.25. This required a very specific sequence of anchor installation and excavation to ensure factors of safety were met at all times. Excavation was limited to elevation 772.0 until the anchors in the cells were stressed. Toe blocks with stressed anchors are provided at the base of the cells to provide an adequate factor of safety against deep-seated failures and tie the weak layers of rock into the more competent strata below. This condition occurs when the rock is excavated 8 ft below the base of the cells at a distance of 2 ft from the cells. Because of the restricted lateral space, the location of the toe blocks had to be carefully chosen and the anchors properly sized. In all cases, foundation pressures were limited to 20 kips.

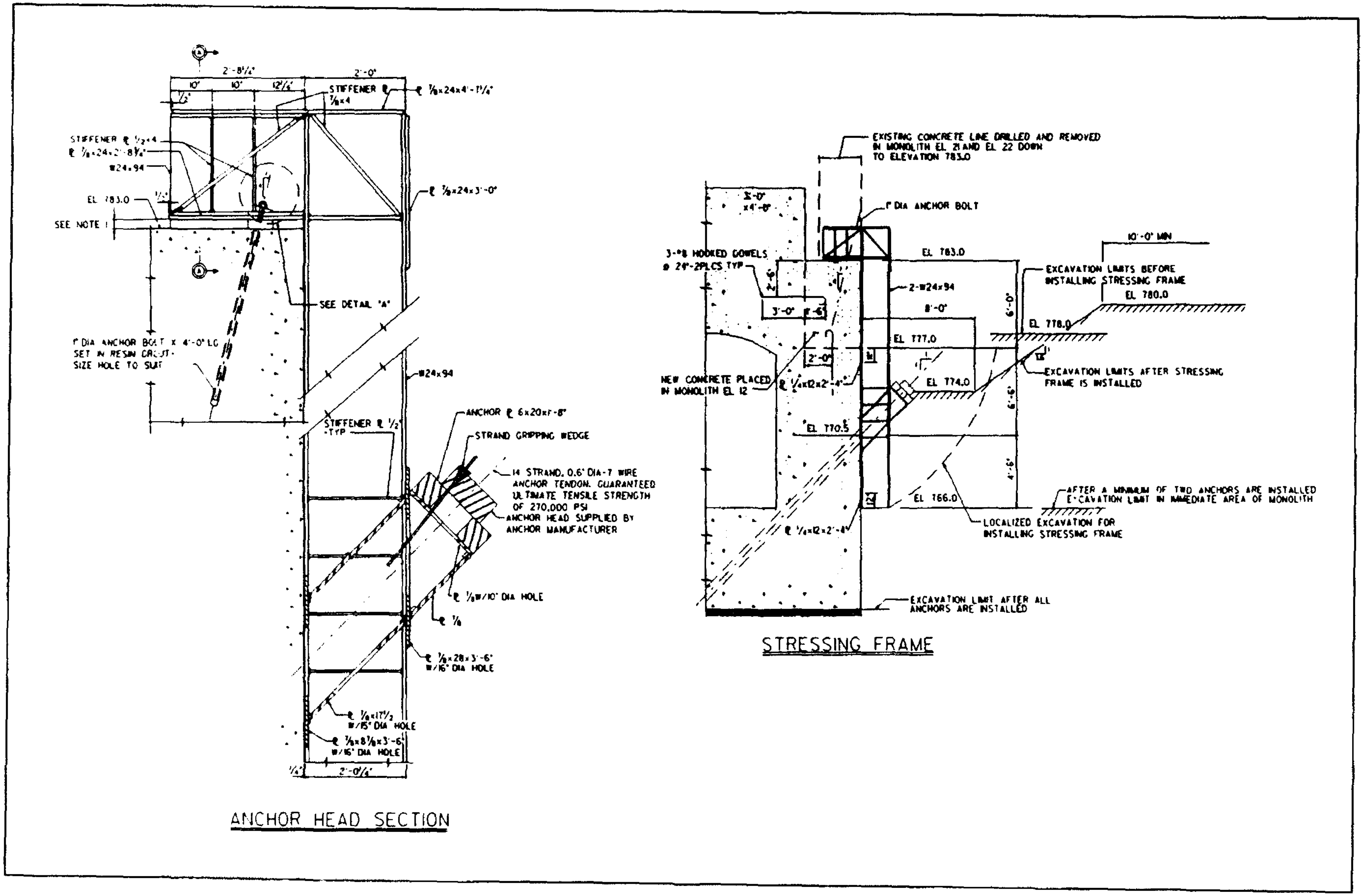

Figure 3. Point Marion Lock cofferdam existing land wall stressing frame

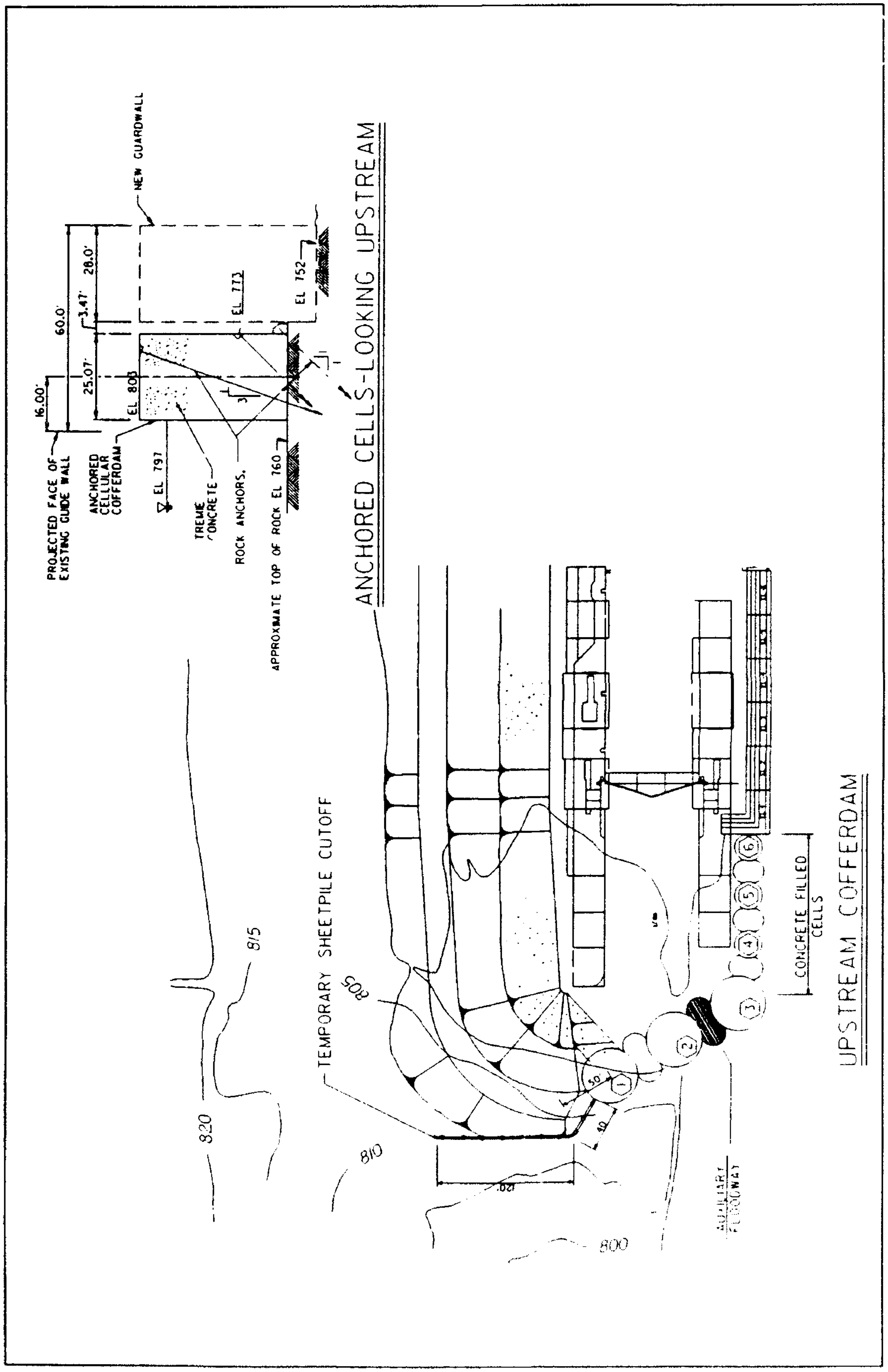

Figure 4. Point Marion Lock cofferdam anchored concrete-filled cells

Design Considerations

The granular-filled cellular cofferdams were assumed to be founded on rock and were designed using Corps program X0040 CCELL. Draft EM 1110-2-2503 was also used in designing the cells. Concrete-filled cells were analyzed as monolithic gravity structures in accordance with ETL 1110-2-22, "Design of Navigation Lock Gravity Walls." An impact load of 1 kip per lineal foot was assumed at normal pool. Sliding was investigated using Corps Program X0075 CSLIDE.

The vertical anchors and some of the inclined anchors are 12-strand, 0.6-in.-diam rock anchors with a working strength of 422 kips and a guaranteed ultimate strength of 703 kips. The majority of the inclined anchors are 14-strand, 0.6-in.-diam rock anchors with a working strength of 492 kips and a guaranteed ultimate strength of 820 kips. The use of a working strength equal to 60 percent of the ultimate strength provides an extra margin of safety. One anchor in each monolith will have an anchor head with a load cell to check for loss of prestress. The vertical anchors are grouted along the secondary length to distribute stresses away from the deteriorated portions of the lockwall face.

The foundation is indurated clay with residual strength values used as a friction angle of 15 deg and no cohesion. Any embedment into rock was ignored. For overturning, each monolith was designed to have 75-percent active base for all loading conditions. The sliding factor of safety was designed to be 1.5 during long-term construction and 1.25 for temporary conditions during installation of the inclined rock anchors. Foundation pressures were limited to 20 ksf. In some cases, the existing computed foundation pressures exceeded 20 ksf before the installation of anchors. In these cases, any reduction in foundation pressures was considered acceptable, even though the resulting pressure exceeded 20 ksf.

Existing Wall Conditions

The condition of the concrete in the existing land wall was a major concern in designing and locating the stressed rock anchors. There was extensive visible damage in the wall faces up to a depth of 8 in. and spalling at the monolith joints up to 12 in. deep. Local areas had damage up to 24 in. deep. Concrete in the area of the miter gate machinery and gate anchorage extended to a depth of 3 ft. The US Army Engineer Waterways Experiment Station conducted a thorough investigation and evaluation of the structure which verified the poor condition of the concrete. Very poor quality concrete was observed in the horizontal and vertical borings taken with the concrete being broken for an average depth of 1.0 and 2.2 ft, respectively. Alkali-silica reaction was present. The lock walls do not contain reinforcing steel or wall armor and were constructed with non-air-entrained concrete. These factors contributed heavily to the spalling and deterioration of the lock wall surfaces. The concrete below the damaged concrete, however, was of good quality. The depth of deterioration in the wall faces contributed greatly to the locations of the vertical anchors. Generally, vertical anchors were installed from 2 to 4.5 ft from the face of the existing land wall. Care had to be exercised in locating anchors between outlet ports and other wall openings. At the back of the wall with the limiting excavation depths, it was necessary to locate anchors 1 ft from the bottom corner of the culvert. This was later revised to 18 in.

Several major cracks were found in the culvert valve monoliths. Five culvert valve and valve bulkhead recess monoliths were tied with 1- 3/8-in.-diam all-thread bars stressed to a working load of 142 kips. These bars, strategically placed, effectively tied together the cracked sections and allow the monoliths to function as a single unit. These bars were installed under close control with restrictions on pool elevation and excavation of backfill to provide the required factors of safety.

Closure of Openings

Three 6- by 4-ft penstock openings in the upper guide wall had to be sealed and closed off to provide a dry cofferdam. These openings were sealed using stiffened steel closure plates with rubber "J" bulb seals. These plates were tied to the inside of the openings' side walls, and the penstocks were then filled with concrete. These closures had to be designed to allow traffic to continue during the construction of the new lock and also to allow the barges to be able to slide along the face of the wall.

Special Investigations

In order to verify the performed traditional analyses and to investigate the possibility of high-stress concentration in the monolith sections and the foundation, the Waterways Experiment Station was engaged to perform finite element analyses. Two monolith sections were investigated, the upper guide wall and a typical land wall monolith for eight separate load cases. In each load case, the pool was held constant to the top of wall elevation 803.0 or 6 ft above normal upper pool elevation 797.0. The first load case represented the existing condition before any excavation. The additional load cases represented the various combinations of anchor loads, intermediate excavation limits, sequencing of anchor installation, and included raising individual anchor loads to 1.25 times the working load. Two additional load cases were run for the existing land wall. These cases were for a flexible foundation analysis of the most critical load case to allow for a comparison of the flexible base and rigid base analyses and also for the dewatered load case on a flexible foundation.

High stress levels were found at the typical land wall monolith for the installation of the upper inclined anchor. High stresses were also found at re-entrant corners. These conditions were considered in the design, and accommodations were made to reduce these stresses. In most conditions analyzed, however, the finite element analysis confirmed results obtained by traditional methods.

Conclusion

This cofferdam arrangment combines innovative design applications and construction sequence requirements to provide a safe, dewatered construction area for a most difficult project. The design involved the use of rock anchor theory, geotechnical considerations, and reinforced concrete design. It involved extensive cooperation of all design elements, including the Ohio River Division office, USACE, and the Waterways Experiment Station.

Melvin Price Locks and Dam Third Stage Temporary Closure

by

James A. Mills, PE[1]

Abstract

The third stage temporary closure was designed to close off the flow through the uncontrolled portion of the Mississippi River between the main lock and the Illinois bank at Melvin Price Locks and Dam. Once the closure was completed, construction of the upstream leg of the third stage cofferdam was able to proceed in slack water. Without the temporary closure, construction of the sheet-pile cells and connecting arcs for the upstream leg of the third stage cofferdam would have become increasingly difficult, due to additional velocities and scouring upstream of the cell construction as the remaining river channel became more constricted with the construction of each additional cell. Closure of the upstream leg of the cofferdam could not be accomplished in a flowing river condition over much of the range of potential flow conditions, thus establishing the need for the temporary closure. This paper covers design and installation of the temporary closure.

Project History

The Melvin Price Locks and Dam project was authorized by Congress to replace Locks and Dam No. 26 on the Mississippi River at Alton, IL, by construction of a new dam, a 1,200-ft main lock and a 600-ft auxiliary lock in three stages.

The first stage of construction was completed in 1984 and consisted of construction of Illinois bank protection, a first stage cofferdam, and six and one-half gatebays of the dam. Navigation was limited to the Illinois side of the river during this phase of the project.

The second stage of construction was completed in 1989 and consisted of construction of a second stage cofferdam, the 1,200-ft main lock, and a half-dam gatebay on each side of the lock. Navigation continued to be limited to the Illinois side of the river during this phase of the project.

The third stage of construction currently underway and scheduled for completion in January 1993 (Figure 1). This phase of the project consists of construction of a third stage cofferdam, a 600-ft auxiliary lock, and one and one-half gatebays of the dam. The third stage cofferdam has been completed and has been unwatered. The auxiliary lock and remaining dam gatebays are currently under construction. The pool was raised in February of 1990, and navigation now passes through the main lock.

[1] Structural Section, US Army Engineer District, St. Louis; St. Louis, MO.

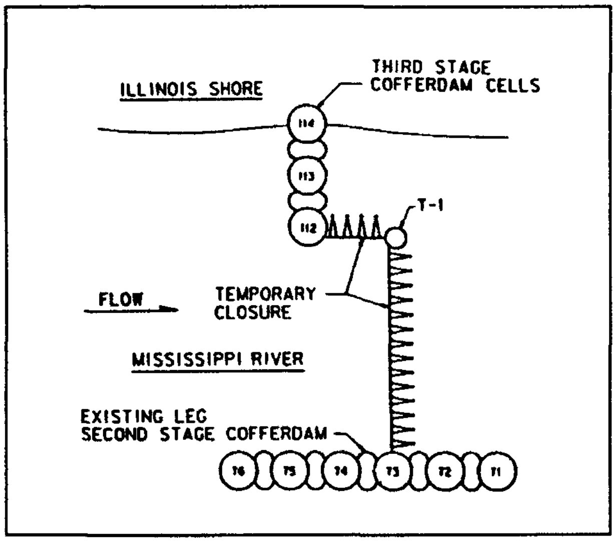

Figure 1. Partial plan, third stage cofferdam

Third Stage Temporary Closure

Introduction

The third stage cofferdam consists of a cellular sheet-pile cofferdam which encloses an area of approximately 25 acres of the Mississippi River channel between the Illinois bank and the 1,200-ft main lock, which is the site for construction of the 600-ft auxiliary lock and remainder of the dam. The third stage temporary closure was designed to close off the flow through the uncontrolled portion of the river between the main lock and the Illinois bank. At the time of construction of the temporary closure, approximately 65 percent of the river's flow passed through this area. The remaining 35 percent passed through the seven dam gates completed during the first two phases of construction. With the construction of each cell of the upstream leg of the third stage cofferdam, the remaining river channel would become more constricted, causing successive cofferdam cells to be constructed in increasing velocities and subjecting the cells to additional scour. Also, as the remaining opening through the upstream cofferdam leg was decreased, the foundation materials for the 600-ft auxiliary lock would be subject to increasing scour. It was concluded that construction of the sheet-pile cofferdam cells and connecting arcs for the upstream leg of the cofferdam could not be accomplished in a flowing river under most of the anticipated riverflow conditions without risking the integrity of the cofferdam and the foundation for the auxiliary lock. Thus, the need for the temporary closure was established to close off the flow so that construction of the upstream leg of the third stage cofferdam could proceed in slack water. Physical model tests were performed by the US Army Engineer Waterways Experiment Station (WES) to determine the construction sequence of the temporary closure and the third stage cofferdam. The model which was used had an undistorted scale of 1:120 and was of the movable bed type. Results from the model tests were incorporated in the contract plans and specifications.

Design

The third stage temporary closure was designed by Booker Associates, Inc., of St. Louis, MO. The closure was designed as steel sheet piles attached to steel bulkhead frames, with the bulkhead units attached to triangular steel support frames connected to 36-in.-diam steel pipe piles.

The design assumed that the loads to the sheet piles would be transferred to the steel bulkheads and that the bulkheads would transmit the loads to the support frames at the frame panel points. The support frames were attached to the steel pipe piles at the top using a moment connection. It was assumed that deflection of the frames under load would cause contact between the bottom of the support frames and the pipe piles transferring the loads to the piles. The lateral loads would be resisted by the pipe piles over their embedded lengths. Three loading conditions were considered:

- Erection and handling loads.
- Loads after installation of the support frames and pipe piles but prior to all bulkheads being in place.
- Loads after the closure was made.

The sheet piles, bulkheads, and frames were analyzed using MCAUTO STRUDL. The support frames were modeled as space frames. The interaction between the pipe piles and the riverbed was modeled using elastic soil springs.

Installation

Construction of the temporary closure began on 10 October 1989 and was completed on 4 November 1989. Fourteen triangular support frames with sheet-pile bulkheads were placed across the remaining channel between Cell 75 of the existing second stage cofferdam and closure abutment Cell T-1 on the Illinois bank. Four support frames with sheet-pile bulkheads were placed parallel to the Illinois bank between Cell T-1 and third stage cofferdam Cell 112. Once this construction was complete, flow through this area was successfully cut off.

The third stage temporary closure was primarily constructed of materials previously used in construction of the second stage upstream deflector and temporary closure, including the PZ 22 sheet piles, the triangular support frames, and the 36-in. steel pipe piles. After the triangular support frames and pipe piles were removed from the second stage cofferdam, they were inspected, repaired, and altered by the third stage contractor, and reinstalled at their current locations in the Mississippi River for construction of the third stage temporary closure. A support frame was lifted from a barge by a single crane, rotated to a vertical position, and set in the river using three lifting points. The support frames were used as templates for driving the 36-in.-diam pipe piles. Each pipe pile had an 8 × 8 tubular pin attached at the top of the pile. The piles were driven to a depth of 35 to 40 ft with the tube bearing down on top of the support frame leg. Erection bolts were used to connect the tube to the support frame leg. The tube was then welded to the support frame leg brackets. The frame leg bracket was designed to allow for misalignment between the pipe pile and the frame leg.

The steel sheet piles were attached to a steel bulkhead framework. Each bulkhead supported 14 panels of PZ 22 sheet pile. Each sheet-pile panel was bolted to the top of the bulkhead waler. A retainer beam opposite each waler held the sheet pile in place during installation. The sheet-pile bulkheads were lifted off a barge by a single crane, hoisted into position, and slide-down guide beams attached to the front of the support frames. Guide pins which were attached to the sheet-pile bulkheads kept the bulkheads aligned with the guide beams. The guide pin assembly was designed to allow the bulkhead to be out of plumb when it was lowered into place while positioning the lower guide pins on the guide beams. A stop attached to the top of the bulkhead allowed the bulkhead to rest on top of the guide beam when the bulkhead was lowered to the proper elevation. The bulkhead stop was then bolted to the top of the support frame guide beam, after which the sheet piles were unbolted from the top waler, driven to a depth of 10 ft, and rebolted to the top waler.

Flexible membrane was used to form the closure between adjacent sheet-pile bulkheads, allowing for considerable misalignment between adjacent sheet-pile bulkheads. The membrane had a standard sheet-pile connection on each side and slid down a mating connection attached to the end of the bulkhead. The flexible membrane was not driven into the river bottom but rested on it after installation.

Conclusion

The third stage cofferdam contract required the contractor to continue work on the temporary closure up to river stage 412.0 National Geodetic Vertical Datum (NGVD). Construction of the temporary closure was accomplished at a river stage of elevation 399.0 NGVD, 17 ft below flood stage. With the help of forces beyond our influence, the temporary closure was constructed under ideal conditions of a low river with low velocities. Hindsight would show this period to be the second year of a

drought period over the Upper Mississippi River and the Missouri River basins. The temporary closure was completed on 4 November 1989. With the temporary closure in place and a low river, the upstream leg of the third stage cofferdam was successfully completed at the end of January 1990. Based on observations during construction and the success of installation experienced under ideal conditions, the temporary closure as designed would have withstood river stages up to elevation 412.0 NGVD.

References

US Army Engineer District, St. Louis. 1985. "Lock and Dam No. 26 (Replacement), Mississippi River, Supplement No. 5 to Design Memorandum No. 2, General Design, Second Lock," St. Louis, MO.

________. 1986. "Lock and Dam No. 26 (Replacement), Mississippi River, Design Memorandum No. 19, Third Stage Cofferdam," St. Louis, MO.

Melvin Price Locks and Dam Lateral Movements of Monoliths

by

Richard R. Sovar[1] *and Thomas J. Quigley*[1]

Abstract

In January 1991, an unusually large separation of an expansion joint was observed on the service bridge deck of the Mel Price Locks and Dam at Alton, IL. This gap was the first indication engineering personnel in the St. Louis District had of a movement of the second stage lock and dam monoliths towards the dewatered third stage cofferdam. This paper presents a history of the project, an analysis of the movement problems, and the implications this experience holds for other large projects which may be constructed with a staged cofferdam sequence.

Project Location and Status

The Melvin Price Locks and Dam project is located on the Mississippi River at Alton, IL, about 30 miles north of St. Louis. The project to date has been constructed in stages, utilizing three different cellular sheet-pile cofferdam configurations. The project construction began in March 1980 with the installation of the first stage cofferdam, which abutted the Missouri shore and extended about a third of the way across the river. The Missouri abutment pier and six intermediate dam piers comprising 6-1/2 gate bays were constructed within the first stage cofferdam. A 1,200-ft lock and two adjacent dam piers were constructed within the second stage cofferdam, which occupied the middle third of the river. A 600-ft lock and two gate bays are being constructed in the third and final stage cofferdam and will tie into the Illinois shore, closing the remaining third of the river. The 1,200-ft lock was opened to navigation in October 1989. Currently, the auxiliary lock and the remainder of the dam are under construction within the confines of the dewatered third stage cofferdam (Figure 1).

Situation

In late December 1990, the St. Louis region experienced a series of major ice storms that deposited approximately 6 in. of sleet over the entire area. River conditions were adversely affected as well, with ice seriously clogging the channel. Operators at Mel Price passed the ice moving in the river through the dam for approximately three straight weeks. This, by St. Louis standards, represented a long period for continuous passage of ice. During the period of ice passage, the river was at a slight flood condition (up to 6 ft over flood stage).

As the ice melted from service bridge deck in late January, an unusually large gap was observed at the location of an expansion joint between the concrete service bridge and pier D-4. Further studies by engineering personnel correlated the gap with an actual movement in the monoliths constructed in the second stage. Inclinometer data from monoliths D-3 and D-4 defined a movement of 7/8 in. toward the dewatered cofferdam and slightly downstream (Figure 2). Visual inspection of the 1-in.

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[1] US Army Engineer District, St. Louis; St. Louis, MO.

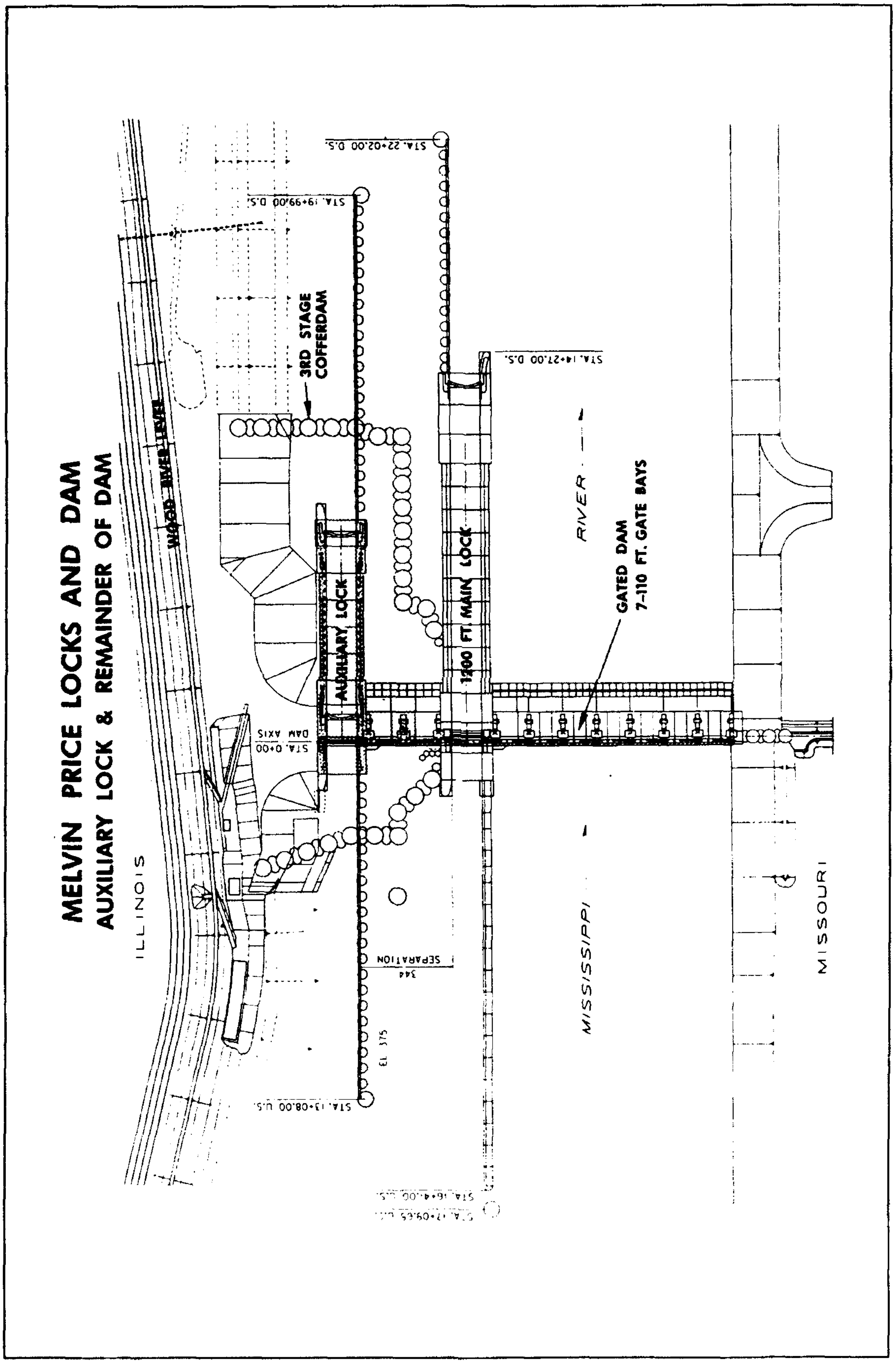

Figure 1. General plan showing third stage construction

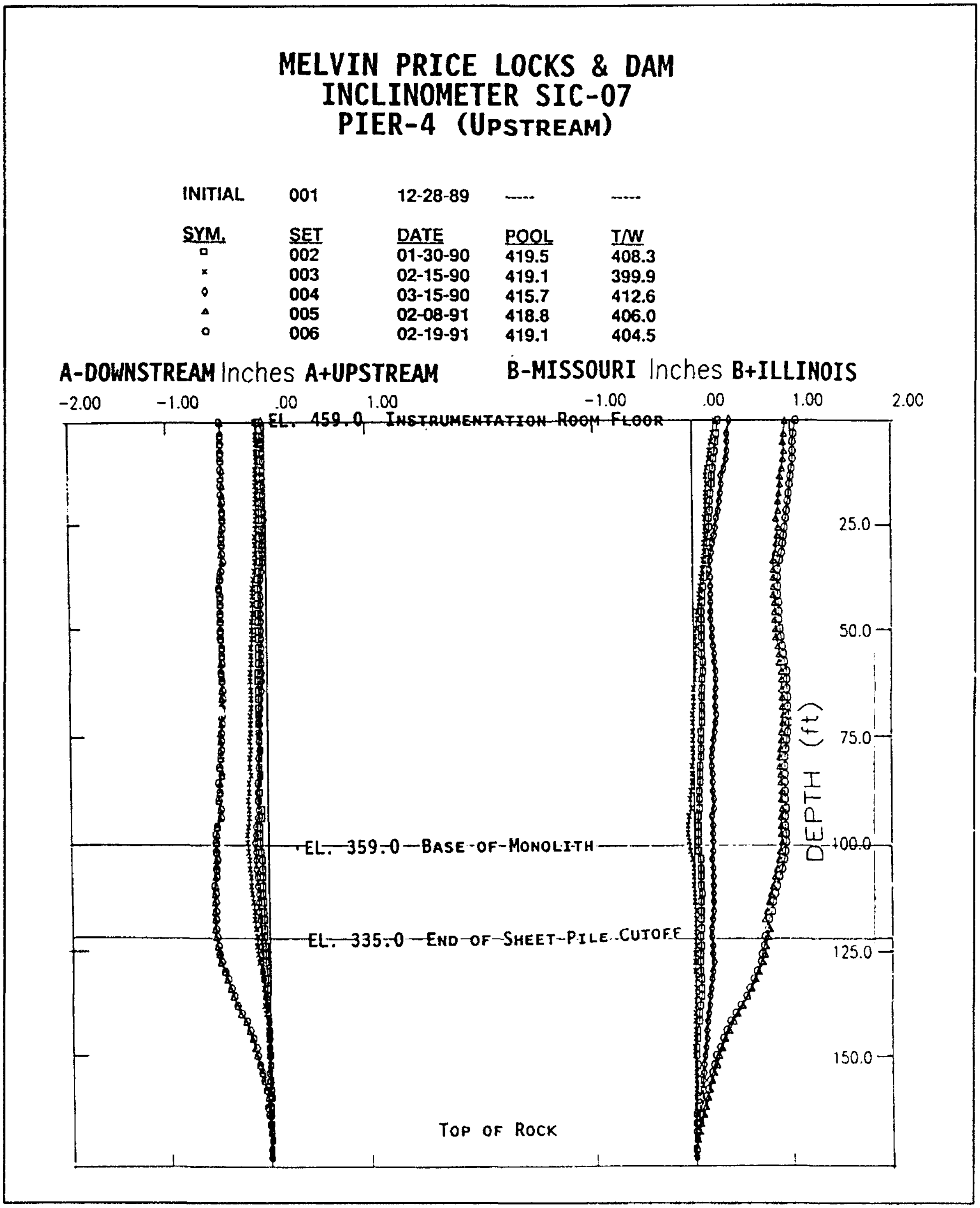

Figure 2. Inclinometer data, Pier D-4 (upstream)

joint filler separating the lock from the L-shaped dam monoliths on each side showed no signs of any compression of the joint. It appeared as if all of the second stage lock and dam monoliths, especially those lock monoliths immediately adjacent to the dam axis, were moving as a group. The movement was concentrated in the foundation between bedrock (tips of the piles at elevation (el) 290) and the bottom of the sheet-pile cutoff at el 335.0 (Figure 3). There has been a deep-seated movement in the zone of the foundation material between rock and the bottom of the sheet-pile cutoff. This paper presents an analysis of the causes of the movements, describes what further studies are ongoing to analytically model the movement, and discusses what implications the movement has for lock and dam designers.

Third Stage Cofferdam

The upstream wall of the third stage cofferdam, composed of a leg of sand-filled circular sheet-pile cells with connecting arcs, originates at the Illinois bank and extends across the old navigation channel to meet the 1,200-ft lock at monolith L-2. From this location to the downstream wall of the cofferdam, the side of six lock monoliths, L-2 through L-7, forms the third stage cofferdamming surface (Figure 4). At monolith L-7, the downstream cofferdam wall, a leg of cells similar to the upstream leg, continues back across the channel and terminates at the Illinois bank. The total area of the dewatered cofferdam is approximately 25 acres.

Upon completion of the upstream leg of the third stage cofferdam in February 1990, the navigation pool was raised and the 1,200-ft lock was put into operation. The lock will be operational during the entire third stage construction period of the auxiliary lock. The cofferdam has been designed to provide construction protection for a 10-year flood frequency. The upstream leg of the cofferdam has a top elevation of 431.0, with sheet-pile tips at varying depths, the deepest being el 326.0. A sand berm 20 ft wide and having a top elevation of 395.0 has been placed along the inside of the entire upstream leg. The downstream leg of the cofferdam has a top elevation of 430.0 with the sheet-pile tips at varying depths, the deepest being el 335.0. A sand berm 20 ft wide and having a top elevation of 385.0 has been placed along the inside of the entire length of the downstream leg. The floor of the cofferdam is at el 365, with construction excavations to el 349.0 for dam pier monoliths D-1 and D-2. There is no sand berm along the concrete monoliths L-2 through L-7 between the upstream and downstream legs of the cofferdam.

Cofferdam Dewatering System

The contractor-designed dewatering system is a "Three Stage Well Suction System" with a temporary stage header at el 380, the main stage header at el 370, and a secondary stage header at el 359. The headers are connected to a series of vertical turbine pump stations.

Construction Within Cofferdam at Time of Movements

Pile driving was the main activity within the cofferdam during the period when the movements occurred. There was also an area of deep rock excavation in the area of D-1 and AL-4 to remove old scour protection that was missed during the rock removal prior to dewatering. The excavation was below the water surface, which at time was approximately el 348.

Analysis of Movement Problems

In order to obtain a better picture of where the movements in the lock and dam structure were occurring, an underwater diver was sent to verify if the gap observed on the bridge deck was indeed occurring between the bases of the monoliths themselves. The affected monoliths have been submerged since the second stage cofferdam was flooded in August 1989. On February 21 and 22 of 1991, an underwater inspection was made of the joints in the dam monoliths located downstream of the tainter gates on the Missouri side of the lock. The underwater inspection verified what visual

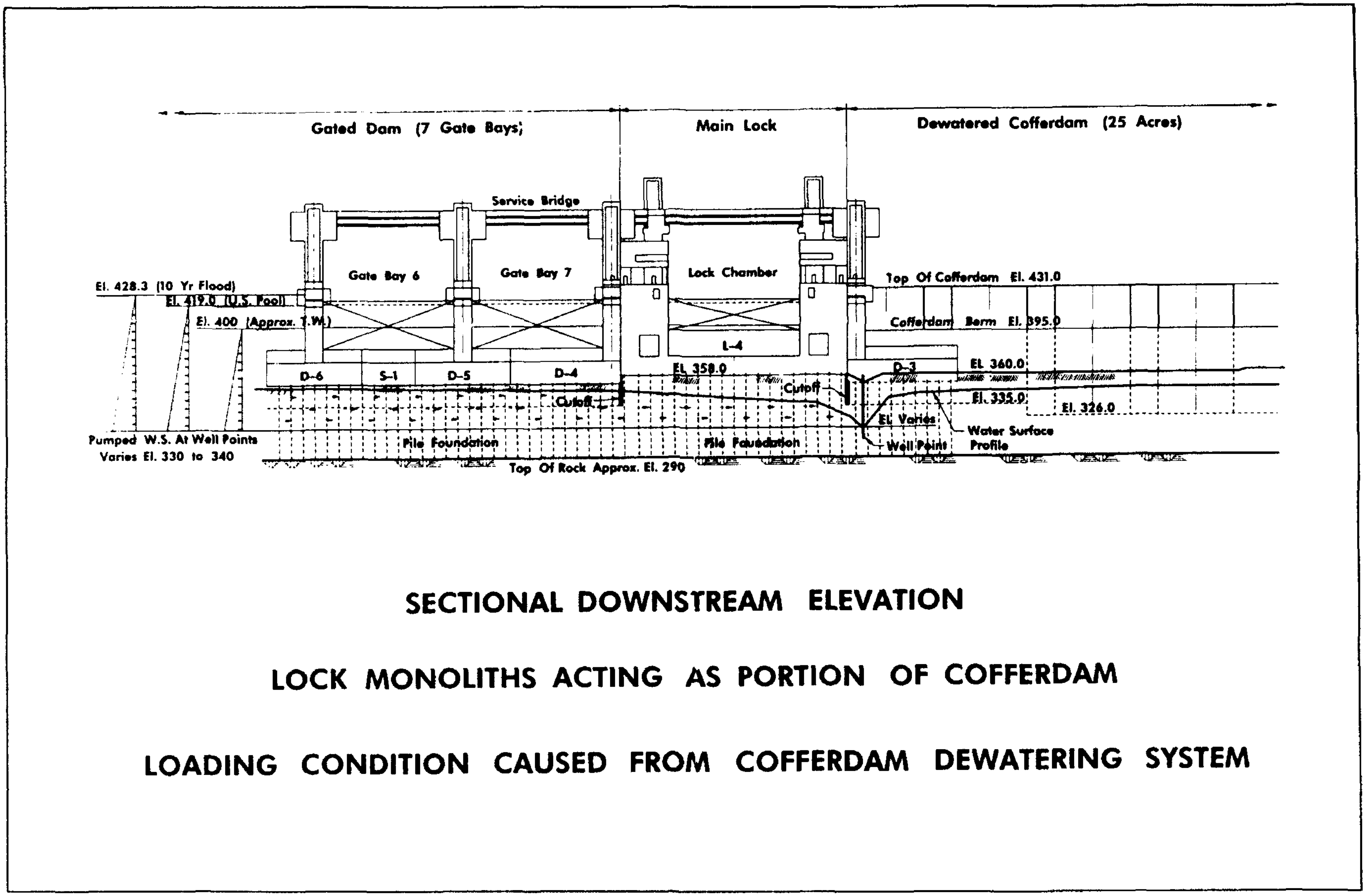

Figure 3. Sectional downstream elevation through movement area

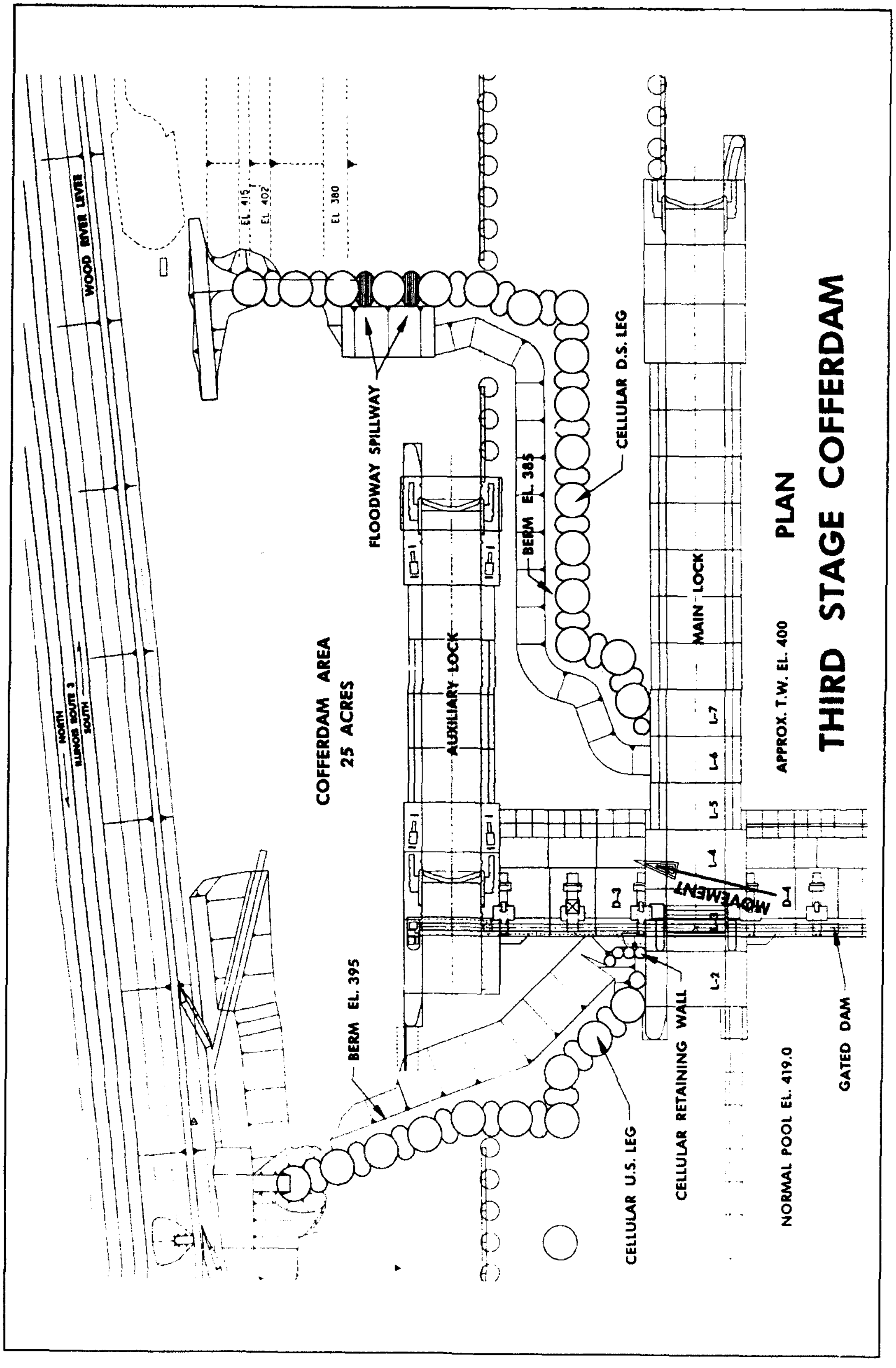

Figure 4. Plan of third stage cofferdam

observations on the surface and instrumentation had indicated, that indeed the monoliths were experiencing a separation. The use of the diver provided us the opportunity to probe the waterstops between the dam monoliths to check their integrity. A broken waterstop would pose the additional complication of loss of foundation material. Fortunately, the waterstops were not broken; however, the investigators experienced some anxiety because the waterstop was not located at the depth shown on the drawings. It turned out that the waterstop was located some 12 to 18 in. lower in elevation than was specified.

That lateral movement would eventually be experienced at Mel Price Locks and Dam was not unexpected. It was the magnitude of the movement experienced that was so surprising. The structural design model of monolith L-3, a flexible base finite element model run with the piles represented as linear elastic springs, had predicted a 3/8-in. lateral movement during normal flow conditions and a 3/4-in. movement when the downstream water elevation was assumed to be at flood conditions (el 428.3). Based on investigation of the movement, St. Louis District engineers believe that monolith L-3 has moved about 3 times what was predicted from the finite element model. In foundation pile layouts for any of the monoliths which moved, L-3, L-4, D-3, or D-4, some of the vertical piles are oriented with their strong axis parallel to the dam axis. No battered piles are oriented in the direction which would resist movement towards the Illinois shore. Because of the number of piles needed to resist the vertical and downstream forces on the monoliths, it was impossible to fit piles which would have been battered in a lateral direction into the layouts.

The exact movement of the monolith is unknown because the District experienced problems in getting baseline trilateration surveys to tie into later surveys. For this reason, St. Louis instrumentation personnel could not pinpoint the movement from a fixed reference. Investigators are measuring gaps in the monoliths as an indication of movement. It is known that the monolith joints contained within the second stage cofferdam were tight before rewatering because a crack survey was made immediately prior to rewatering. Figure 5 shows the movement detected in the monolith joints as a result by both visual inspection and underwater diver inspection.

Following the discovery of the movement, the District called in two experts to assist in the evaluation of what caused the movement and, more importantly, to evaluate whether, in their opinion, further movement could be expected. These advisors were Dr. Lymon Reese of the University of Texas at Austin and Dr. Reed Mosher from Waterways Experiment Station, who came to St. Louis on 4 March 1991. They were briefed on the problem and the history of the movement by personnel from the St. Louis Engineering Division. It was the opinion of the District investigators that the movement was a result of the following factors:

- The static lateral water pressure exerted on the monoliths from the unbalanced water loads on each side of the cofferdammed structures.
- Static lateral water forces on the sheet-pile cutoffs and foundation materials, unaccounted for in the design, due to the large influence of the dewatering system. These forces were not accounted for in the design, which assumed that the concrete monoliths only experienced a lateral water load on the base of the concrete (see Figures 3 and 6).
- Vibration of the lock and dam monoliths due to the passing of ice during an unusually long period in January and February 1991.
- Vibration of the lock and dam monoliths due to bearing pile driving operations associated with the third stage construction within the dewatered cofferdam and the use of a vibratory compactor on sands within the third stage cofferdam.

In the March 4 meeting, both Dr. Reese and Dr. Mosher agreed with the preliminary

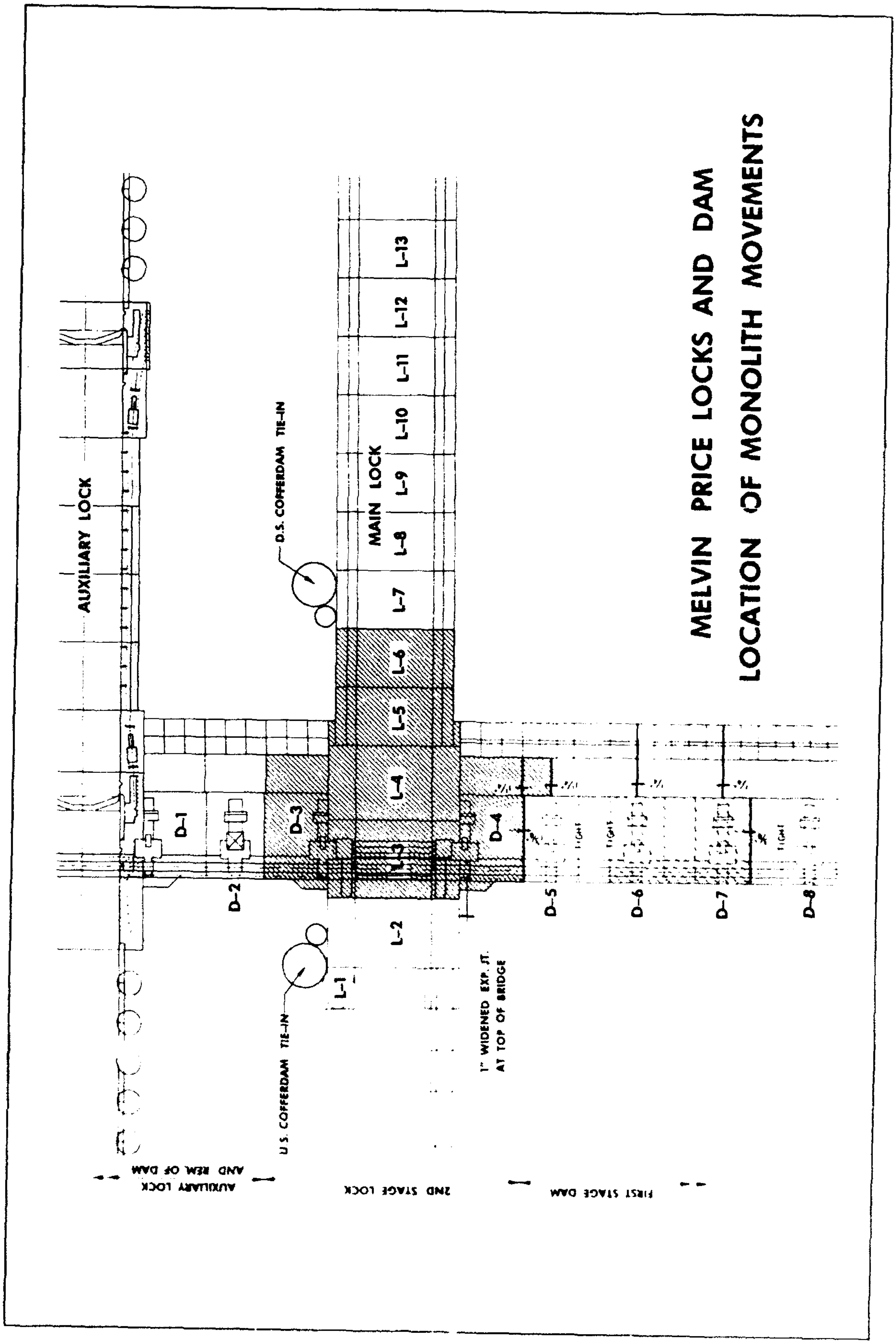

Figure 5. Location of monolith movements

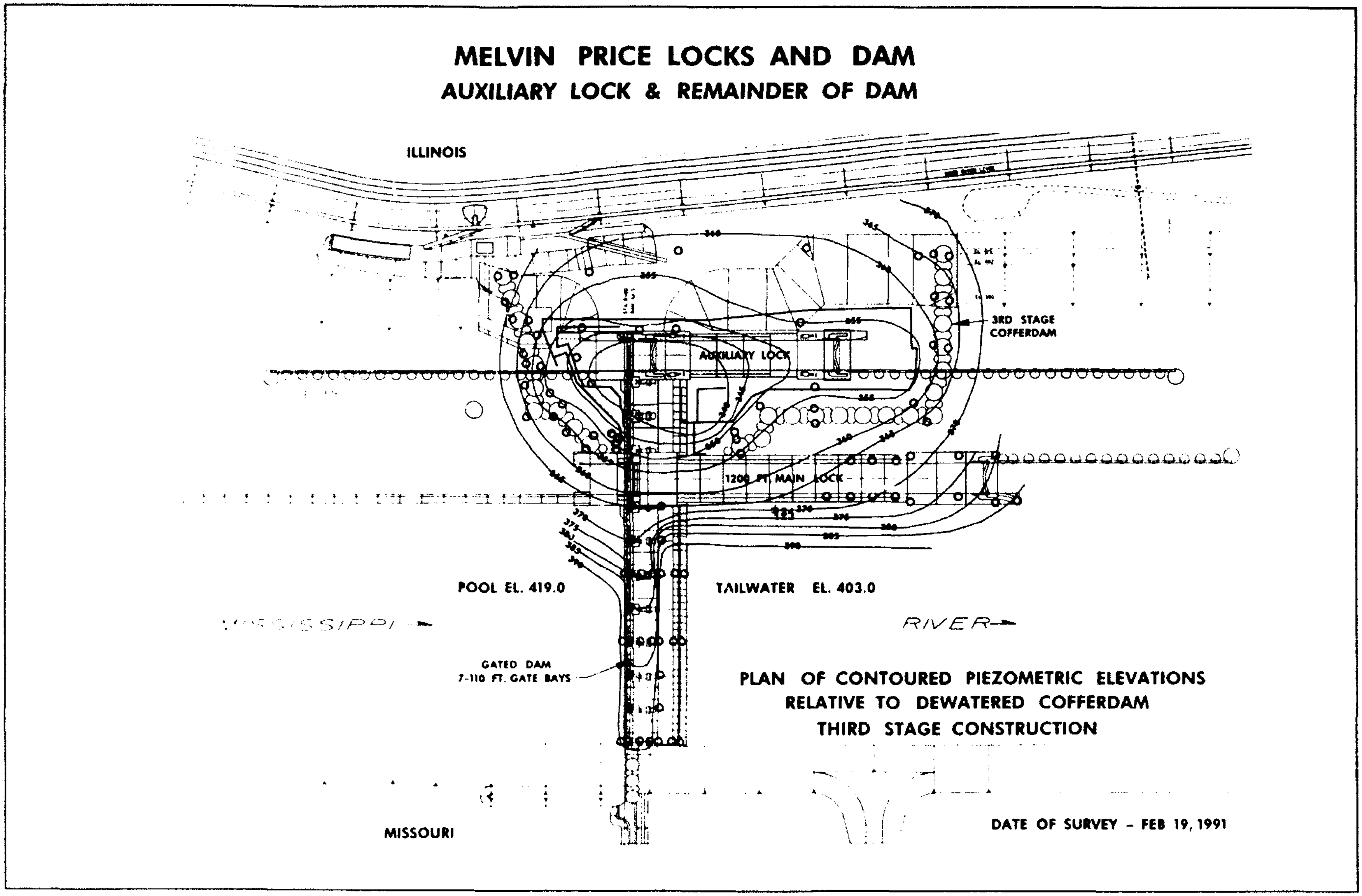

Figure 6. Plan of contoured piezometric elevations

conclusions of the District as to possible causes for the movement. They also felt that the District need not worry about further movement, because of the following factors which were tending to help the monoliths in resisting movement:

- No further ice flow could be expected until the concrete bases of the monoliths adjacent to the dam axis (D-2, D-1, AL-2, and AL-3) were completed.
- All pile driving within the cofferdammed area was essentially complete.
- With the completion of the lower concrete placements on monoliths D-2 and D-1 (to el 365), the forms could be removed, and the water elevation within the cofferdammed area raised from el 350 to el 355.
- The first lift of concrete above the bearing piles is being placed or has been placed for monoliths D-2, D-1, and AL-3, thus providing additional piling to resist the lateral load towards the Illinois shore.

On 2 April 1991, the St. Louis District received a proposal by Dr. Mosher of WES for a further study of the monolith movement. Dr. Mosher believes the most likely cause of the lateral movement towards the dewatered third stage cofferdam is the seepage forces applied to the foundation due to the dewatering. The seepage force is a result of the frictional drag, or pulling force, on the individual soil particles in the foundation resulting from water migration towards the dewatering headers. Dr. Mosher's proposal is to build a two-dimensional finite element model of the base of the dam and lock monoliths and their foundations. The model will coincide with the axis of the dam and will include dam monoliths D-6 through D-3 and lock monolith L-4. The piling will be modeled as a beam with an equivalent stiffness per foot of pile. The computer program SOILSTRUCT, developed by Dr. G. Wayne Clough of Virginia Polytechnic Institute, will be used to run the model.

Implications of This Experience

The engineering manual dealing with pile foundations, EM-1110-2-2906, dated 15 January 1991, cautions the designer that, "Depending upon the type of foundation material, the nature of the loading, the location of the ground water, and the functional requirements of the structure, a detailed seepage analysis and/or pile settlement analysis may be required to define adequately the pile-soil load transfer mechanism and the resulting parameter necessary for an adequate pile design." The St. Louis District feels that a very thorough and extensive analysis of the pile foundation was done at the time of design, which included a deep- seated analysis of the foundation. The District also believes the zone of influence of the dewatering system was underestimated. This dewatering system was designed by the contractor through a performance type of specification. The contractor was instructed that he would have to maintain the water level 10 ft below the lowest point in the cofferdam in which he was working. It was not anticipated that, during the construction of the auxiliary lock, the contact surface between the bottom of the lock floor and the soil foundation would remain dry beneath the entire lock monolith L-3 or L-4 in a direction along the dam axis.

In developing computer models of the lock monoliths which served as the cofferdamming surface, District structural engineers applied a lateral water load to the concrete base of these structures, and further down on the sheet-pile walls. However, the actual uplift was less than originally assumed thereby increasing the lateral load on the sheetpiling. On dam monolith D-4, as well as the stilling basin monoliths immediately downstream of it, there was essentially no differential horizonal water load applied towards the Illinois shore, as it was assumed that the effects of the dewatering system would not extend that far from the headers. Also, the frictional force caused by the movement of water through the sand particles beneath monoliths D-4, D-3, and the lock monoliths which comprised the third

stage cofferdamming surface, towards the dewatering headers buried in the third stage cofferdam, was not considered.

It is still uncertain how much effect vibrations due to the ice passing in combination with the pile driving added to the problem. In addition to Dr. Mosher's study, the District is also considering funding a study of the effects of vibration in inducing lateral movement in pile-founded structures.

At Mel Price Locks and Dam, the St. Louis District was very fortunate. The movement was not large enough to rupture the waterstops between the monoliths nor to cause any significant operational problems with the tainter gate in the bay where the movement was concentrated. It is hoped that what has happened in St. Louis will be an educational experience furthering the Corps' understanding of similar structures to Mel Price which are constructed on piling. When the studies described are complete, the results will be available from the authors of this paper.

Criteria Update Related To TM 5-855-1

by
William H. Gaube, PE[1]

Abstract

The current criteria update initiatives of the Protective Design-Mandatory Center of Expertise (PD-MCX) parallel a comprehensive research program being conducted jointly by the Army, Air Force, and the Defense Nuclear Agency (DNA). Criteria subjects related to TM 5-855-1, "Fundamentals of Protective Design for Conventional Weapons" (Headquarters, Department of the Army, 1986) include in-structure shock, ground shock prediction, and penetration of munitions. Criteria subjects related to both conventional weapons and accidental explosions include spallation of concrete, passive airblast attenuation, the BLASTX computer program for prediction of airblast inside rooms, and new Corps of Engineers Guide Specifications (CEGS) covering blast resistant valves and doors.

Introduction

This paper discusses current criteria update initiatives of the PD-MCX related to conventional weapons resistant design. This criteria update effort parallels a comprehensive research program in conventional weapon resistant design being conducted jointly by the Army, Air Force, and DNA. The criteria update items discussed cover fiscal year 1991 only. However, this criteria update program is expected to continue in subsequent fiscal years. The ultimate goal is a complete update of TM 5-855-1, "Fundamentals of Protective Design for Conventional Weapons" (Headquarters, Department of the Army, 1986).

In-Structure Shock and Shock Isolation

In-structure shock and shock isolation is one of the first subjects being addressed by the current research in conventional weapon resistant design and is probably the most deficient area in the 1986 version of TM 5-855-1.

Design methodology

The methods for predicting in-structure shock and designing shock isolation systems will be considerably expanded. Some potential improvements are discussed.

- **Structure motions.** The analysis of structure motions will include both a simple free-field modification method and the use of discrete structure models. The free-field modification method will provide simple procedures for transforming free-field ground motions into structure motions and will be similar to the methodology presented in the current manual. The discussion of discrete models will cover the analysis of flexible structures using multidegree-of-freedom analysis.

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- **Shock spectra.** The discussion of shock spectra will cover the formulation of shock spectra using both the simple amplification method and shock spectra calculation. In the amplification method, shock spectra are determined using constant amplification factors applied to the structure motions. Shock spectra calculation involves the multiple calculation of peak motions over various system frequencies.

- **Equipment shock tolerance.** New equipment shock tolerance data will be presented. This data will improve equipment assessment and result in more rational decisions on the level of shock isolation to be provided.

- **Shock isolation configurations.** Information on the analysis of rigid mass systems will be added. This information will focus on the analysis of base-mounted and suspended pendulum systems.

- **Shock isolation devices.** Information will be added on the selection and characterization of isolation devices such as helical cable isolators (cable mounts), elastomeric isolators, and other devices.

- **Isolation details.** Guidance will be added on details such as the requirements for equipment connections to piping and ducts, isolator to platform connections, and the design of isolated platforms.

- **Hard mounting.** More comprehensive guidance will be provided on hard mounting equipment that does not require shock isolation.

Computer programs

A major obstacle to proper shock isolation design has been the lack of computational tools required to perform the analysis. The criteria update program will help to eliminate this deficiency through the development of the new PC-based computer programs described in the following paragraphs.

- **Structure motion.** This computer program will calculate motions for two-dimensional reinforced concrete structures loaded by airblast or ground shock.

- **Shock spectra.** This computer program will calculate shock spectra. The program features tripartite comparison plots between calculated shock spectra and a self-contained library of equipment shock tolerance data.

- **Configuration analysis.** This computer program calculates the motion response of a rigid mass system supported by shock isolators and excited by structure motions.

Ground Shock and Penetration of Munitions

Ground shock

Ground shock results when a conventional bomb or other weapon detonates at or below the ground surface. The accurate prediction of ground shock is important because many facilities designed to resist conventional weapons are built below grade.

During the past few years considerable research has been conducted in the area of ground shock prediction. These research activities have included the following:

- Full-scale controlled backfill tests to measure ground shock in soil materials having known material properties.

- Development of a computer automated data base of constitutive properties from past test data. This data base characterizes soil material behavior for large strains and strain rates.

- Two-dimensional analytical calculations that encompass weapon detonation (weapon burn), cavity formation, ground shock, and crater formation.

- One-dimensional analytical calculations of free-field ground shock to determine the character of the ground shock waveform.

The research will result in the complete revision of the ground shock prediction methods in TM 5-855-1, Chapter 5. The new ground shock prediction methods will include better waveform models and a better prediction of waveform parameters.

Penetration of munitions

The penetration of munitions is becoming increasingly important. The effectiveness of penetrating weapons was recently demonstrated in the Kuwaiti conflict when coalition forces used heavy guided penetrators to destroy enemy aircraft shelters and command centers.

Research activities in antipenetration measures have included tests on concrete materials, cemented and noncemented rock boulder layers, and steel frame yaw inducing screens that turn projectiles sideways so that deep penetrations cannot occur. This research will result in an update to the penetration prediction method presented in TM 5-855-1, Chapter 4.

Other Protective Design Criteria

The discussion thus far has concerned protective design criteria related almost exclusively to structures designed to resist the effects of conventional weapons. The remaining criteria update items apply to structures designed to resist the effects of both conventional weapons and accidental explosions.

Spallation of concrete

Concrete spallation caused by airblast and fragment impacts can be controlled by the methods indicated below.

- Construction of an earth berm on the blast side of concrete walls.
- Installing a steel spall plate on the wall surface opposite the blast.
- Building thicker concrete walls.

The criteria update covering concrete spallation is concentrated on wall thickness. The approach is to use both available test data and analytical calculations to better understand the spall mechanisms, identify the parameters that most affect spall behavior, and develop a simple design aid for concrete thickness selection.

Passive airblast attenuation

There is mounting evidence that the short-duration airblast created by high explosives can be effectively attenuated by passive systems rather than the active blast valve systems currently used. The passive airblast attenuation concept has been undergoing extensive testing since 1989. Design guidance on passive airblast attenuation will be developed as soon as testing is completed and evaluated.

BLASTX Computer Program

BLASTX is a computer program for the prediction of airblast in rectangular rooms due to an explosion in the room or the inflow of blast overpressure from an adjacent room. This program is currently being enhanced by incorporating new data entry menus and plot reporting.

Blast Valve Guide Specification

A new CEGS covering active blast valves will soon be issued. The new blast valve CEGS covers blast overpressures, valve operating mechanisms, operating temperatures, ventilation air flow, testing, and installation.

Blast Door Guide Specification

A new CEGS covering manually operated, side swinging blast doors is currently under development. The new blast door CEGS covers structural steel, reinforced concrete, and hollow metal doors. This CEGS will contain information on contractor blast design, door hardware, fabrication, shop and field testing, and installation.

Revision of the Tri-Services Design Manual, "Structures to Resist the Effects of Accidental Explosions" (TM 5-1300, NAVFAC P-397, AFM 88-22)

by
Paul M. LaHoud, PE[1]

Abstract

Design of hardened structures to resist the effects of accidental explosions must provide protection which complies with specific criteria defined in Department of Defense explosive safety regulations, DoD 6055.9-Std (Department of Defense 1984). The implementation these criteria in structural design were first formalized in 1969 in the Tri-Service Design Manual, "Structures to Resist the Effects of Accidental Explosions" (Headquarters, Departments of the Army, Navy, and Air Force 1969). The procedures provided in this manual concentrated on reinforced concrete structures, although they were eventually applied to other materials. Since its original publication, a large amount of additional research data and design experience have been accumulated. This led to a decision in 1981 to revise the manual and incorporate new data. This paper describes the general changes as well as some of the more specific additions to its contents.

Introduction

Design of facilities against blast effects produced by accidental deflagration/detonation of conventional explosive materials, constituent feedstocks, and high pressure/high temperature materials in processing, manufacturing, or storage is a highly specialized field of structural design. The first official Department of Defense (DoD) design guidance was furnished in 1969 when the design manual "Structures to Resist the Effects of Accidental Explosions" (Headquarters, Departments of the Army, Navy, and Air Force, 1969) was published as a Tri-Service Manual. The significance of this manual was that it provided, for the first time, Department of Defense Explosive Safety Board (DDESB) approved procedures which could be used to design structures to provide protection for personnel, equipment, and facilities. Protection requirements in processing, manufacturing, transporting, and storing explosives are much more stringent than in other military applications. Figure 1 describes the arena of application of TM 5-1300 (Headquarters, Departments of the Army, Navy, and Air Force, 1969). Figure 2 shows the specific Army and Air Force safety regulations requiring application of the manual.

While the 1969 version of the manual represented a landmark advance in quantification and standardization of design guidance, it was developed primarily for reinforced concrete structures. In the intervening years, large amounts of new research, design, and construction experience accumulated. The need for design guidance for materials other than

[1] US Army Engineer Division, Huntsville; Huntsville, AL.

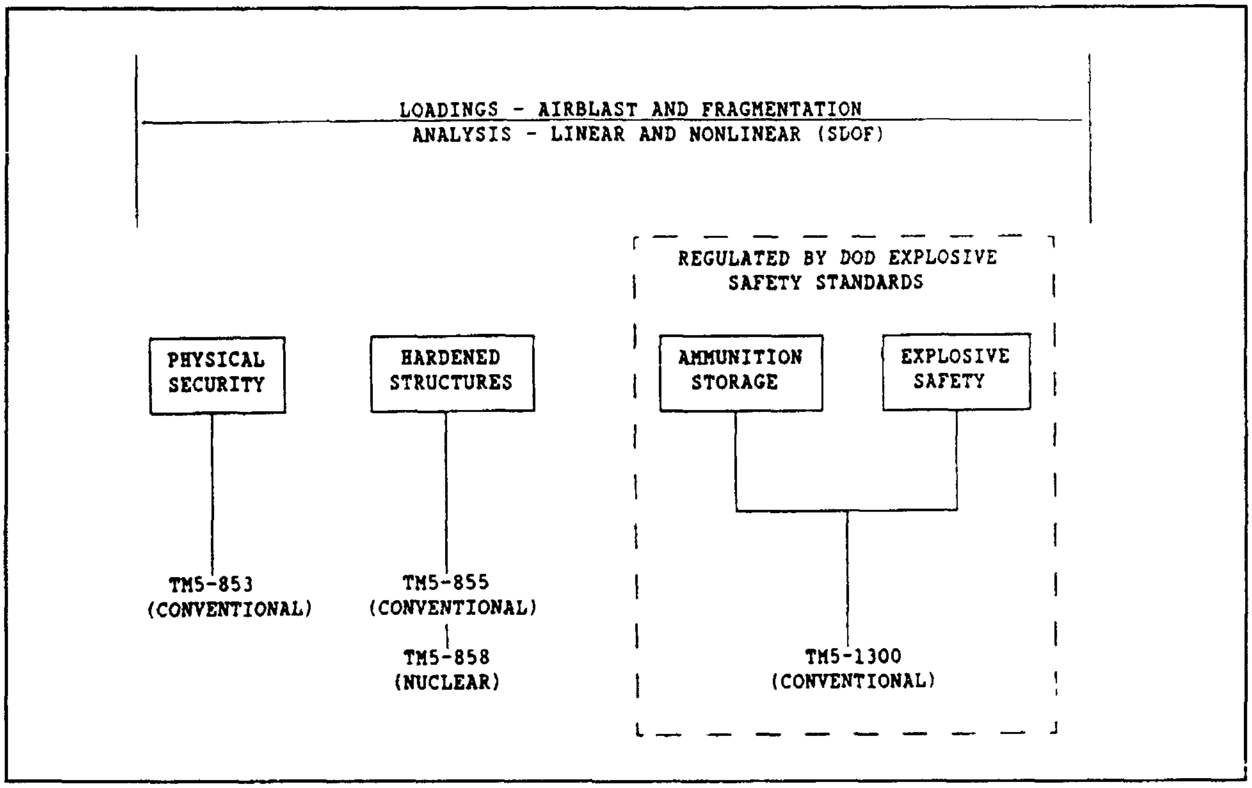


Figure 1. TM 5-1300 design applications

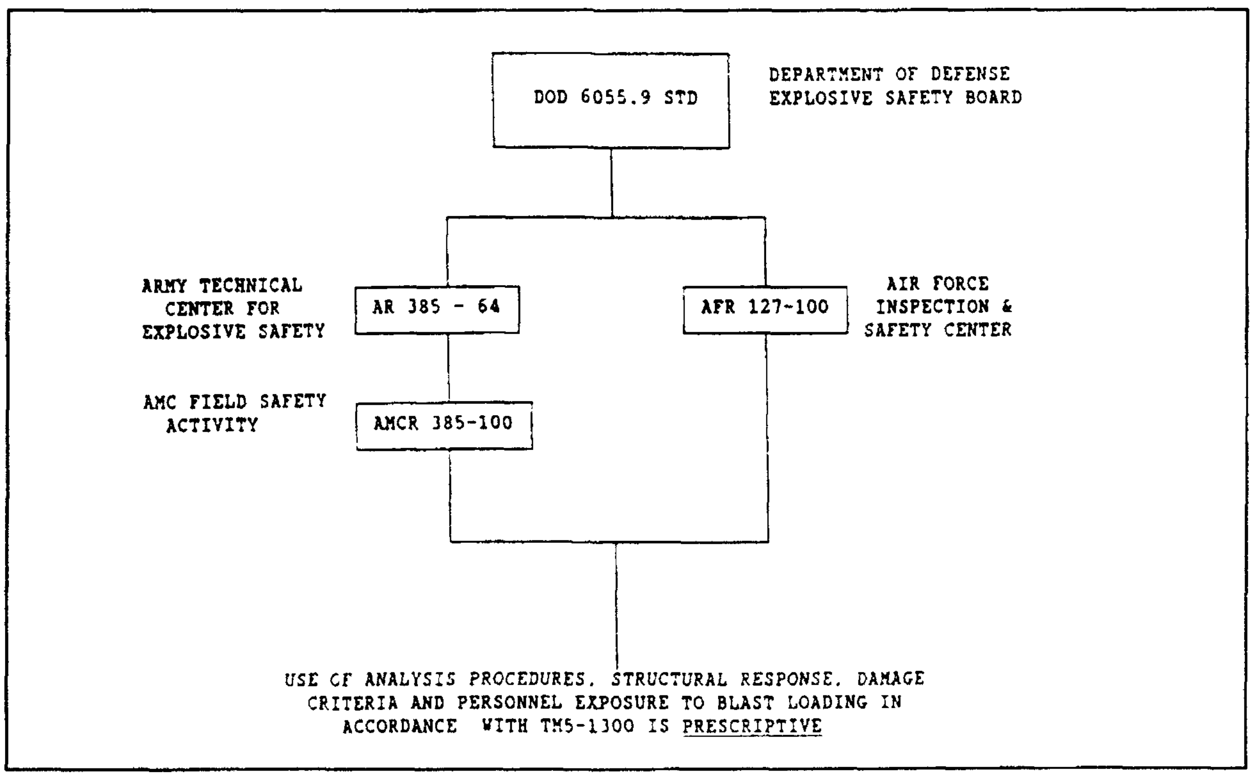


Figure 2. Governing explosive safety standards

reinforced concrete also became more evident. In 1981 a decision was reached to initiate a revision to the manual. The principal management and funding thrust for this revision effort originated with the US Army Armament Research and Development Center (ARDEC). ARDEC recognized the need for new design technology for process and storage facilities to support the ammunition development and manufacturing mission of the Army Material Command (AMC) Munitions Production Base Programs. The DDESB also contributed funding, technical, and management support.

The revision effort to the manual was initially guided by a steering committee with subcommittees for blast technology and design applications. In 1987 with most of the research completed and the draft manual released for technical review, the management structure was streamlined to a combined management/ technical steering group. Figure 3 shows the historical evolution of the committee. The current steering group will continue to manage the manual and periodic revisions.

The revision to TM 5-1300 "Structures to Resist the Effects of Accidental Explosions" (Headquarters, Departments of the Army, Navy, and Air Force, 1969) represents a significant increase in physical size from the original manual as demonstrated in Figure 4. One of the most valuable features of the original manual was the presence of example design problems. The new manual follows the same philosophy throughout all six volumes and provides many examples demonstrating both original and new material. This paper will discuss the contents of each of these volumes compared with the original manual. Examples of significant changes will be briefly discussed.

Volume I-Introduction

The material contained in Volume I consists of an expanded discussion of the topics contained in Chapters 1, 2, and 3 of the original manual. Significant additional data and discussion are provided on the topics of human tolerance to both blast overpressures and shock, explosive initiation by fragments, and equipment tolerances to shock loads. Figure 5 shows an example of information on human tolerance to overpressure. Valuable new information on safe separation distance for numerous types of munitions is also included. The contents expands from 7 pp in the original manual to 42 pp in the new manual. Additional technical references provided increases from 8 to 17 pp.

Volume II-Blast, Fragmentation, and Shock Loads

This volume esentially supersedes Chapter 4 of the original manual and demonstrates the abundance of new data acquired between 1969 and 1987. Total content on blast, fragment, and shock loading increases from 65 pp in Chapter 4 of the original manual to over 500 pages in Volume II of the revised document. Source references increase from 25 to 157. A significant new topic discussed is the effect of explosive source geometry on blast pressure and impulse. This is an important concern in estimating the energetic output of explosive processing and manufacturing operations. Figure 6 shows examples of source geometries. Figure 7 is one example of 32 pp of new data on this topic obtained from various ARDEC and AMC and Department of Energy (DOE) sources.

Another topic which has benefitted from extensive new research is the calculation of blast loads on cubicle walls. The original manual provides charts to calculate only the impulse loads on cubicle walls with various boundary conditions. Subsequent research and analyses have shown in many cases a pressure-time solution may be required rather than an impulse solution. In addition to the original impulse charts, 38 new charts are provided which allow the calculation of the average reflected pressure on the same cubicle walls.

Extensive Naval Civil Engineering Laboratory (NCEL) research on blast load environments in and adjacent to fully and partially vented cubicles has been incorporated into the new manual. Thirty-two additional new

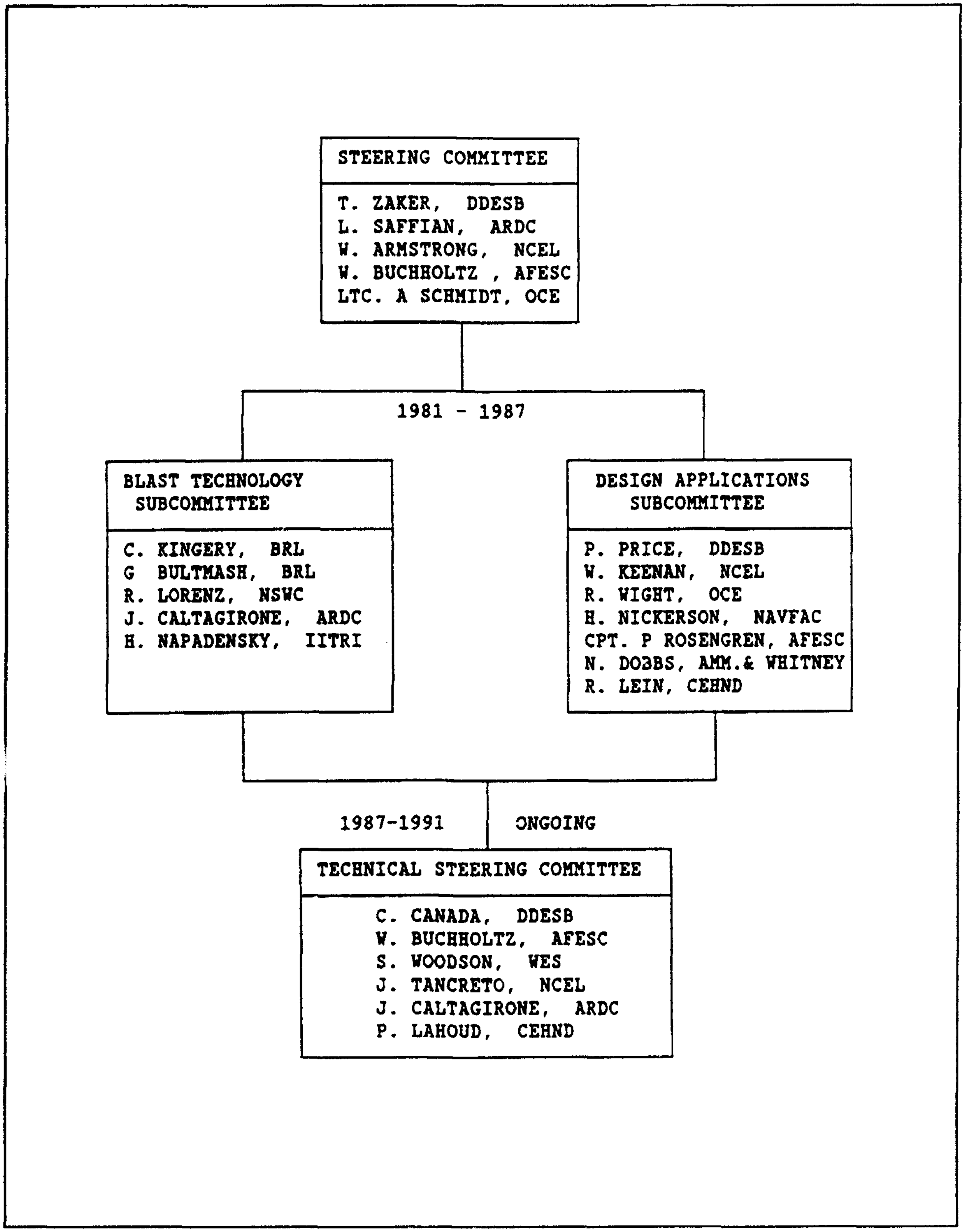


Figure 3. Manual steering committee

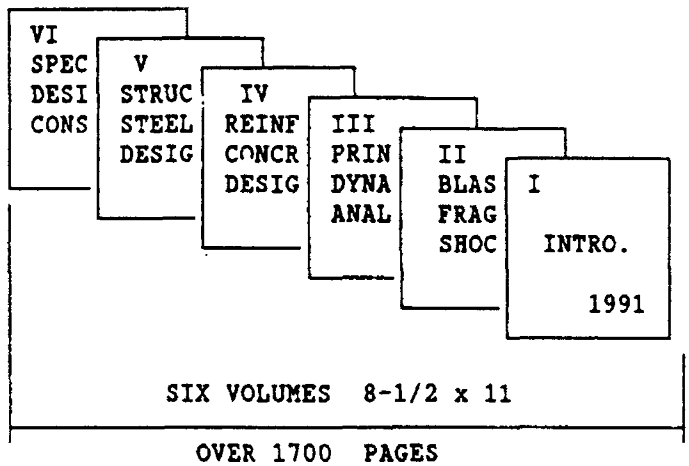
THE OLD

TM 5 - 1300

SINGLE VOLUME 11 X 17

188 PAGES 1969

THE NEW



VOLUME	I	INTPODUCTION
VOLUME	II	BLAST, FRAGMENTATION AND SHOCK LOADS
VOLUME	III	PRINCIPLES OF DYNAMIC ANALYSIS
VOLUME	IV	REINFORCED CONCRETE DESIGN
VOLUME	V	STRUCTURAL STEEL DESIGN
VOLUME	VI	SPECIAL CONSIDERATIONS IN EXPLOSIVE FACILITY DESIGN

Figure 4. Comparison of old and new versions of TM 5-1300 (Departments of the Army, Navy, and Air Force, 1969)

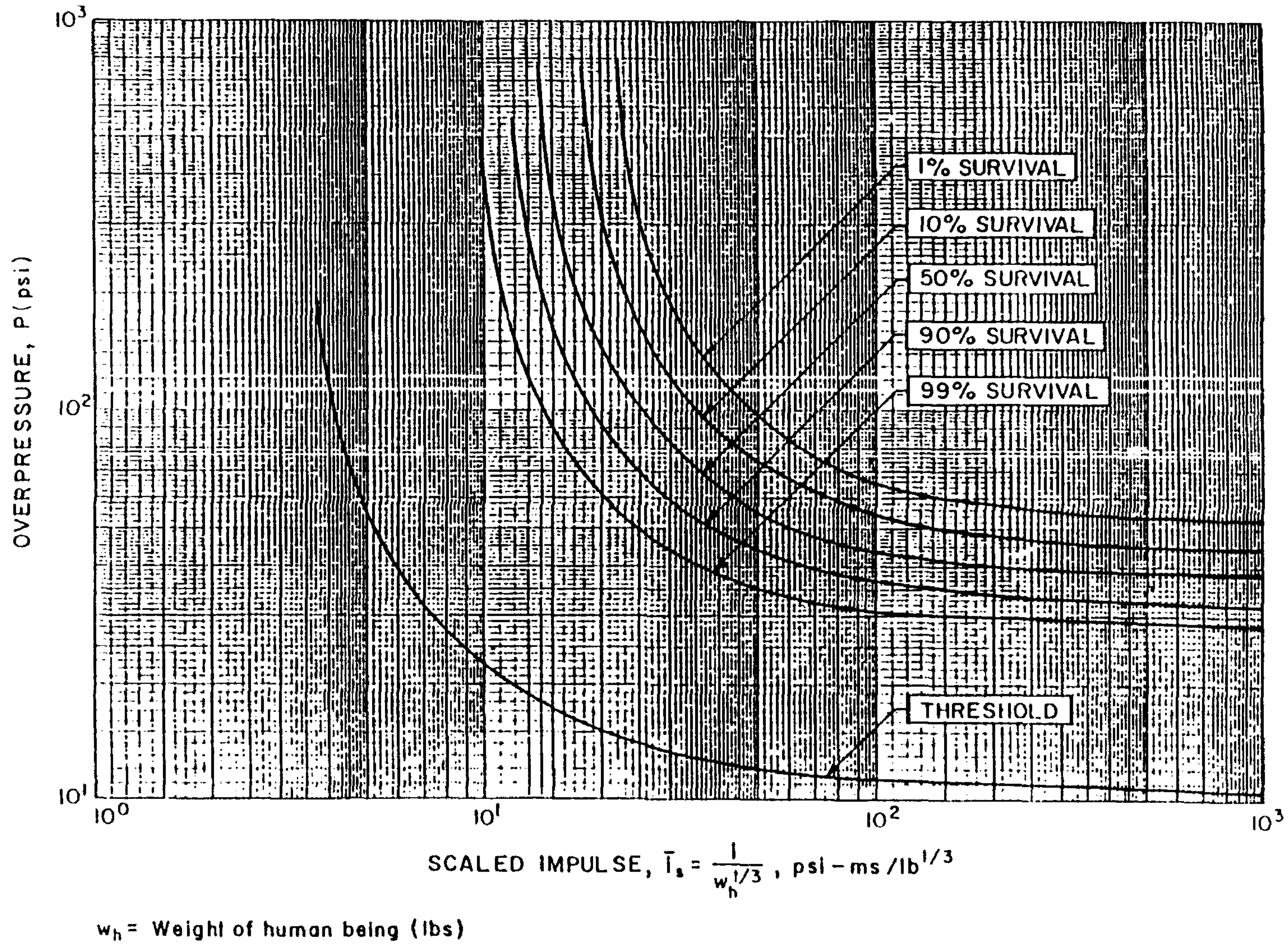


Figure 5. Survival curves for lung damage

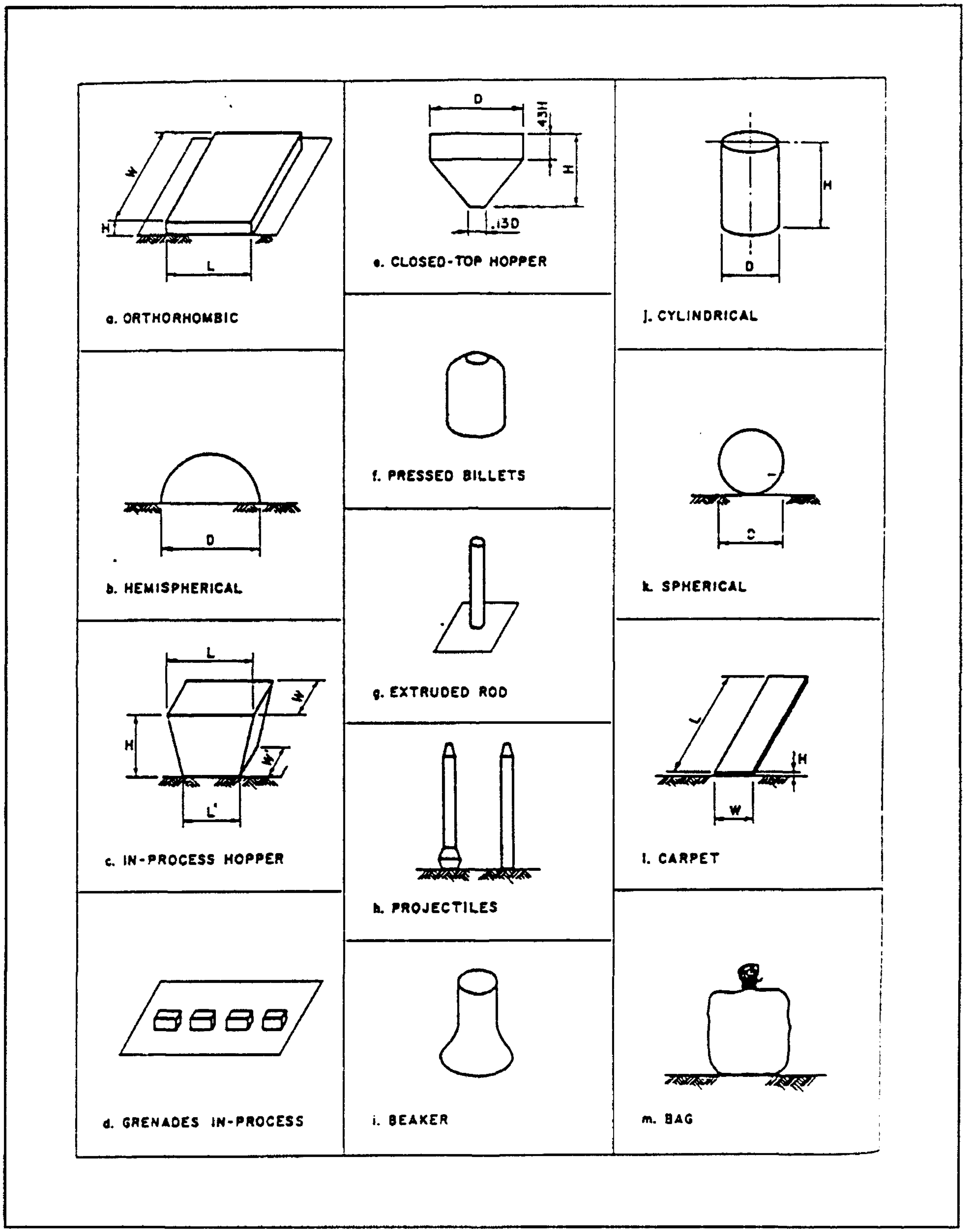


Figure 6. Explosive shapes

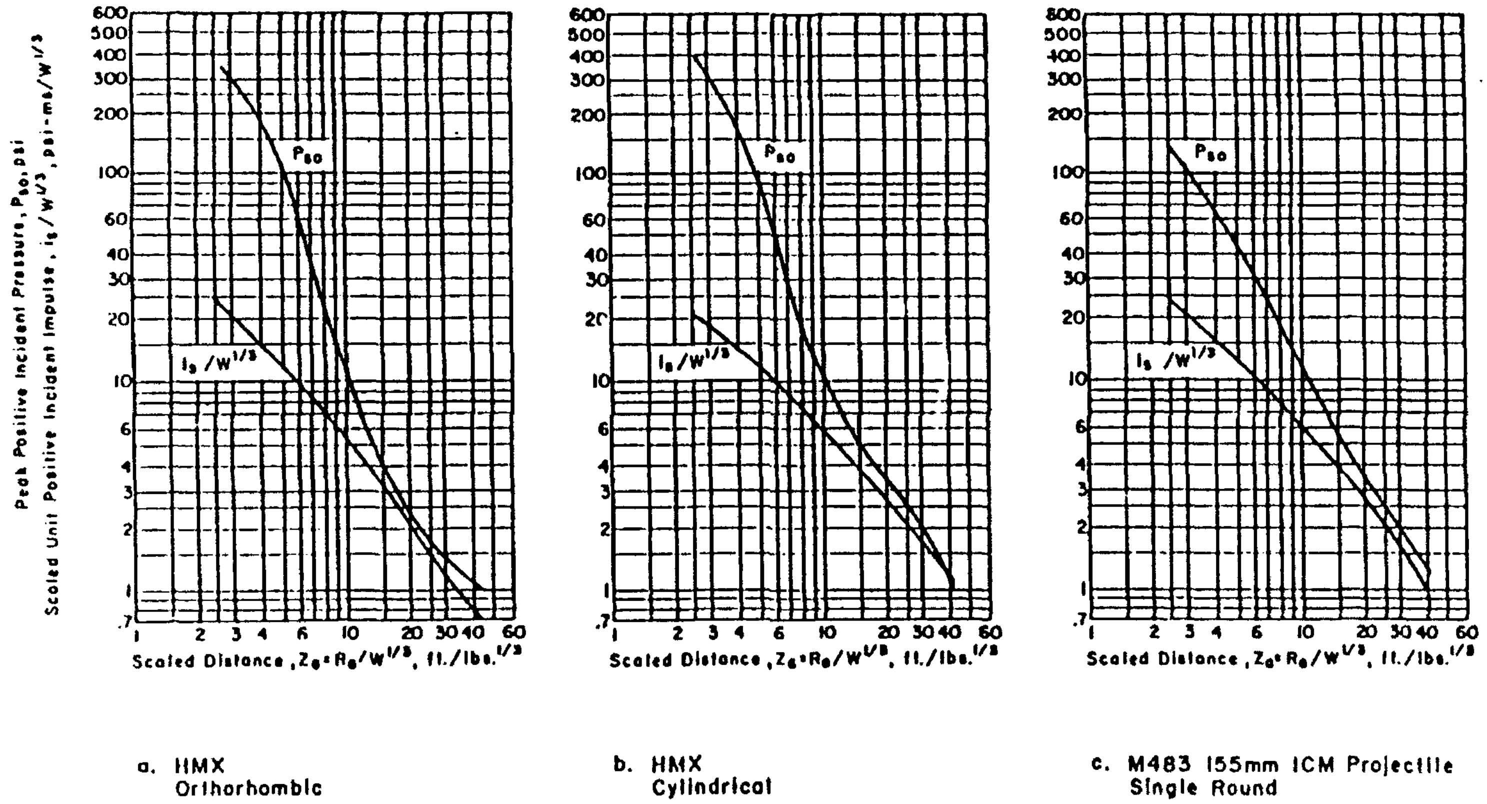


Figure 7. Peak positive incident pressure and scaled impulse for an explosion on the surface at sea level

charts are provided which allow the estimation of gas pressures in, and shock pressures adjacent to, various configurations of cubicles. This is a common design problem in hazardous process manufacturing facilities.

Fragmentation prediction procedures in the original manual were limited, addressing only primary fragment effects from conventional explosive filled munitions. More detailed analytical models for primary fragmentation have been developed and incorporated based on ARDEC work. These procedures allow the estimation of primary fragment mass and velocity for noncylindrical geometries. Additional experimental and analytical material on primary fragmentation from liquid-filled munitions developed from the chemical weapons demilitarization program (US Army Engineer Division, Huntsville, 1983) has also been included. Volume II of the new manual incorporates extensive additional analysis procedures from DOE (US Army Engineer Division, Huntsville, 1980) on prediction of secondary fragmentation of equipment and building elements.

Volume II also incorporates procedures for the estimation of groundshock from accidental explosions. This data was extracted from TM 5-855-1 (Headquarters, Departments of the Army, Navy, and Air Force, 1986). Methods are presented for determining the motions caused by ground shock and air blast effects as well as their interaction. Other procedures are presented for determining shock spectra which may be used for evaluation of structure motion as well as the design of shock isolation systems.

Another topic which has received considerable additional treatment is the calculation of gas pressures as a contributor to total loading from internal explosions in buildings, a very common concern in processing of explosive materials. Recent research has shown that even with frangible walls or vents, gas pressure is a much larger contributor to total loading than had been previously been assumed.

Expanded methods and examples are provided for the calculation of exterior loads on structures as well as interior loads on structures due to leakage of exterior blast pressures through openings. This is very valuable in the design of shelter-type structures where personnel protection from overpressure is required.

Volume III-Principles of Dynamic Analysis

In the original manual basic principles of dynamic analysis were provided as subparagraphs of Chapters 5 and 6. The new manual has reorganized this material in a single volume and extensively supplemented and expanded the methods presented to cover a more complete range of possible structural response situations. Material provided has increased from approximately 50 pp in the original manual to over 375 pp in the new manual.

Data for determining resistance-deflection functions and yield line locations have been significantly increased in the new manual. This new material includes the determination of elastic and elasto-plastic moment and deflection coefficients for various support and loading conditions, including both one- and two-way elements, as well as flat slabs.

As in the original manual and other most widely used hardened structures design references, the new manual utilizes single-degree-of-freedom (SDOF) methods to estimate the maximum response of structures subjected to blast loads. Only two design charts were provided in the original manual for determining structure to blast overpressures. One chart pertained to structure response to direct loading while the second was used to determine rebound forces. The number of design charts furnished in Volume III has been increased to 216 and these charts cover maximum elastic response to triangular loads, rectangular loads, gradually applied loads, triangular pulse loads, and sinusoidal loadings. The new charts also cover maximum response to elasto-plastic systems including triangular loads, rectangular loads, gradually applied loads, triangular pulse loads, and bilinear-triangular loads, as well as rebound forces.

A beneficial new addition to Volume III includes procedures for performing numerical integration analyses. These include both the average-acceleration-method as well as the acceleration-impulse-extrapolation-method. Procedures are presented for including damping in a system as well as for analyzing two-degree-of-freedom systems. These procedures are much more attractive with the availability of microcomputer spreadsheets with graphics, such as LOTUS 123, and they provide a very flexible analysis tool.

Volume IV-Reinforced Concrete Design

The technical data from Chapters 5 through 9 of the original manual have been combined in this volume. The original manual was concerned primarily with the design of laced reinforced concrete walls to resist the effects of close-in detonations. A considerable amount of new data have been added to address other types of concrete elements. Less than 90 pp of material in the original manual has been increased to over 250 pp in this volume. References cited have increased from 38 to 77. This additional data will facilitate the design of a wider range of reinforced concrete structures.

The new manual provides better procedures for the estimation of the dynamic capacity of both concrete and reinforcing steel. Based on recent research and testing, the dynamic increase factors for both concrete and reinforcing steel are presented as a function of the actual response of the structural elements as well as the values needed for design. In addition, the static yield strength of the reinforcement is increased 10 percent beyond the minimum specified by the American Society for Testing and Materials (ASTM) to account for the actual strength steel that is furnished by steel producers. Finally, the procedures for the determination of shear capacity have been significantly revised (Ross and Zehrt 1990).

New material has been provided for small deflections (less than 2-deg support rotation) design of slabs reinforced with single-leg stirrups rather than lacing. This type of shear reinforcement will greatly simplify construction and result in considerable cost savings.

This volume also provides greatly expanded design procedures for conventionally reinforced slabs and walls of various support conditions, as well as design procedures and deflection criteria for beams and both interior and exterior columns. The design of slabs includes not only one- and two-way slabs of various support conditions, but also flat slabs. Also, when support conditions permit, tension membrane action of the slabs is incorporated in the design. The recognition of membrane action permits the slab to attain relatively large deflections at reduced strength, thereby achieving greater economy in design.

Data on spalling of concrete have been increased to more realistically predict the need for costly structural steel spall plates. In addition, material on structural response to primary and secondary fragment impact is expanded.

The last portion of this volume greatly expands the design detailing procedures from the original manual. These details are expanded to include information acquired from numerous construction projects. Detailing procedures are provided for laced concrete elements, conventionally reinforced concrete, elements incorporating either single leg stirrups or lacing, flat slabs, beams columns, and foundations. Figures 8 and 9 are typical of details provided.

Volume V-Structural Steel Design

The material provided in this volume is entirely new since the original manual did not address structural steel at all. Technical Report 4837 (Picatinny Arsenal 1975), prepared by Ammann & Whitney, New York, provided sources of the procedures in this volume. Most of the design procedures follow from classical SDOF analysis procedures. Mechanical properties of structural steel elements are presented, along with recommended dynamic design stresses and acceptable maximum displacement, and plastic deformations within the broad range of steels available. The structural steels

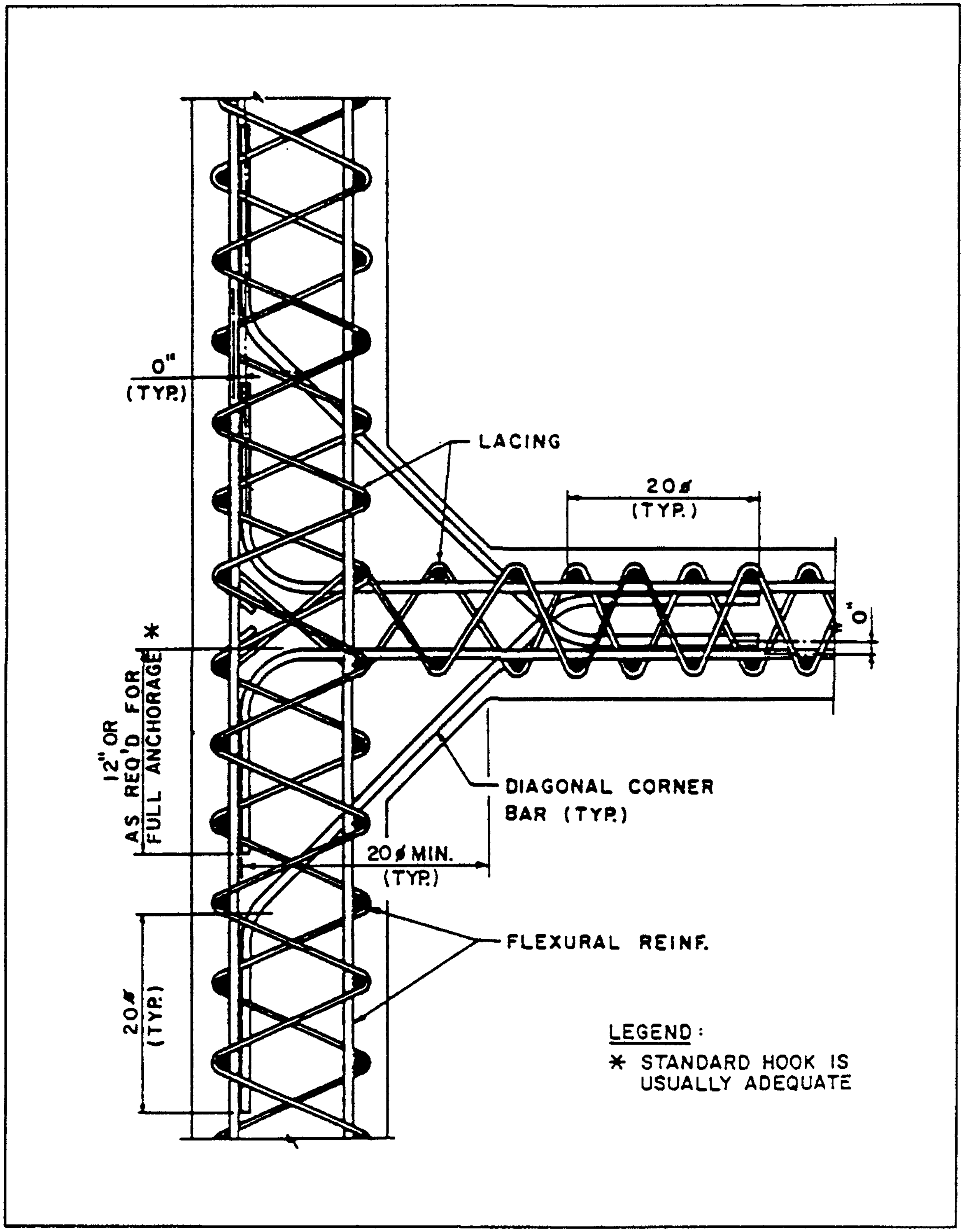


Figure 8. Intersection of continuous and discontinuous laced walls without extensions

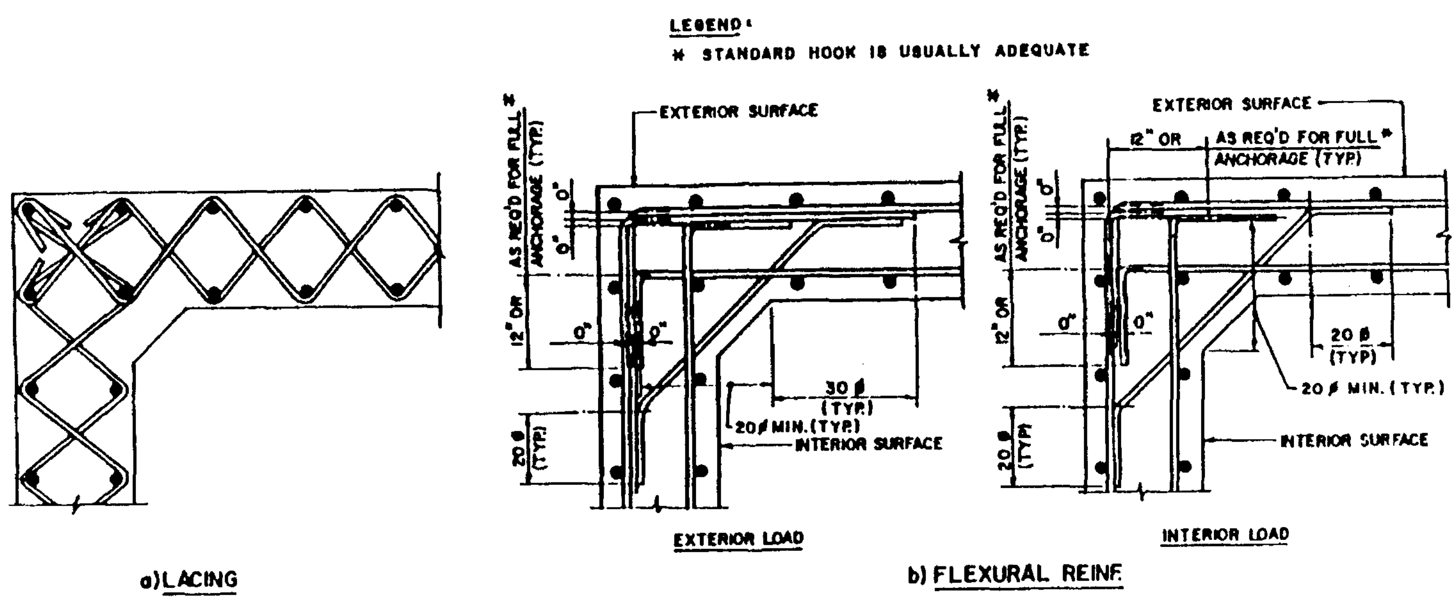


Figure 9. Corner details for laced walls without wall extensions

for plastic design covered by the American Institute of Steel Construction (AISC) specifications are reviewed with regard to their uses in protective structures subjected to blast loads. The effects of rapidly applied dynamic loads on the mechanical properties of steel as a structural material are considered.

A method for performing preliminary blast load design of structural steel frames is presented. This analysis requires the determination of the flexural capacities of individual members. The analytical procedures can consider both single and multibay arrangements for both rigid and braced frames. Based on the results of the preliminary analysis, a final frame analysis can be performed. A section is also provided which outlines methods of detailing connections for structural steel.

Volume VI-Special Considerations in Explosive Facility Design

Chapter 10 of the original manual discussed several miscellaneous topics related to explosive safety protective construction. This volume includes that data and other new data on a wide list of topics including: (1) masonry design; (2) precast concrete design; (3) special provisions for preengineered buildings; (4) suppressive shielding; (5) blast resistant windows; (6) design loads for underground structures; (7) earth-covered arch-type magazines; (8) blast valves; and (9) shock isolation systems.

Masonry design

This section describes the procedures for design of masonry walls subjected to blast overpressures. Methods are included for construction of blast-resistant masonry walls. Included in the design procedures are methods for calculating the ultimate strength of masonry walls as well as resistant-deflection functions. Design criteria are presented for allowable deflections.

Precast concrete design

Described in this section are the procedures used for design of precast elements subjected to blast overpressures. Methods are included for construction of precast concrete slabs, beams, and columns. The design procedures include methods for calculating ultimate resistance and resistance-deflection functions as well as deflection criteria. Procedures are presented for analyzing precast elements.

Special provisions for preengineered buildings

Standard preengineered buildings are usually designed for conventional loads (live, snow, wind loads, etc.). Blast-resistant preengineered buildings are also designed in the same manner as standard structures. However, conventional loadings, which are used for blast resistant design, must be somewhat larger than conventional loadings to compensate for the effects of the blast loads. This section presents the magnitude of these larger conventional loads as well as present details of both the main frame members and foundations which must be incorporated into the building design. Also presented is a recommended specification for preengineered buildings which are to be hardened.

Suppressive shielding

Presented is a summary of design and construction procedures which are outlined in the Design Manual, "Suppressive Shields-Structural Design and Analysis Handbook," HNDM 1110-1-2, (USAE Division, Huntsville, 1977). This section describes the application of suppressive shielding as well as design criteria and procedures. Methods of designing equipment penetrations through walls, as well as blast doors to be used with suppressive shielding, are discussed.

Blast resistant windows

Historically, explosion effects have produced airborne glass fragments from failed windows which are a risk to life and property. Guidelines are presented for the design, evaluation, and certification of windows to safely survive a prescribed blast environment. Design criteria is presented for both glazing and window frames. The design procedures include a series of design charts for both the glazing and frames.

Design loads for underground structures

This section is a summary of the data presented in the Design manual, "Fundamentals of Protective Design for Conventional Weapons" (Headquarters, Department of the Army, 1986). The data contained in this manual pertains primarily to effects produced by explosions on or below ground surface and the blast pressures they produce on below-ground structures. Procedures are presented for evaluating blast loads acting on the structure surface as well as structure motions caused by explosions.

Earth-covered arch-type magazines

This section deals with typical earth-covered magazines which are used for storage of explosives.

Included are requirements for both metal arch and reinforced concrete arch magazines, including semicircular and oval types. Discussed are the investigations performed in connection with magazines, design procedures, construction, and standard designs. Additional information is available from Williams, Farsoun, and Watanabe (1990).

Shock isolation systems

Data presented for shock isolation systems have been greatly expanded from that given in the present manual. The data given in the original manual were basically qualitative rather than quantitative. Although a full discussion of the subject is beyond the scope of this volume, an introduction to isolation system design is presented. Included are various methods of achieving shock isolation for both equipment and personnel. Typical designs for equipment supports are presented.

Blast valves

This section discusses several types of blast valves that are available commercially, including sand filters, hardened louvers, and poppet valves. Also presented are the advantages and disadvantages of blast-actuated vs remote-actuated blast valves, the effect of plenum chambers, and a sample set of specifications for the design, testing, and installation of a poppet valve. More current research in this area is now under joint development by the Protective Design Mandatory Center of Expertise (PD-MCX) and the US Army Engineer Waterway Experiment Station (WES) including passive valves and may be available in the near future.

Computer Analysis Procedures

TM 5-1300 is recognized and applied worldwide. One of the main objectives of the steering committee was to assure that the new manual provided complete methods suitable for manual hand calculations. This resulted in the large number of new charts and graphs suitable for direct design. However many valuable computer codes have been developed to solve segments of the analysis process. Codes which are currently approved by the DDESB for application of methods in this manual include:

CBARCS/PCBARCS	SOLVER
FRANGE	SCHOCK
TRAJECT	FRAGHAZ

Other codes, if used, must provide benchmark problems to demonstrate compliance with the data in the manual. Numerous codes under development by the PD-MCX, including BLASTIN, TUNREF, and SPIDS, will be

evaluated in the future for conformance with manual criteria and approval for design.

Each of the three services will maintain repositories which contain computer programs that are consistent with the procedures and techniques contained in the new manual. The sources are: WES, NCEL, and Wright Patterson AFB. A limited number of copies of the programs will be available from each repository upon request. The individual programs will be identical at each repository. The manual steering committee will continue to evaluate new codes and requirements for revisions to existing codes. These programs will be periodically updated or revised as required.

Summary

The new version of TM 5-1300 received Tri-Services approval in early 1991 and is awaiting publication. The manual is greatly respected worldwide, and many nations use it as a basis for their own explosive safety guidance. Within the United States, numerous other government agencies direct its application including DOE and NASA. While not yet published, its full contents are available in a superb microcomputer interactive manual developed by Mr. Dave Hyde at WES for the DDESB. This automated version includes all tests, graphs, charts, and many time-saving analysis algorithms. It will be familiar to those of you that have used the automated version of TM 5-855-1, also, developed by Mr. Hyde. However, every wonderful new advance has its price. The TM 5-1300 automated manual requires approximately 20 megabytes of hard disk space. However, to me it looks like an opportunity to justify that 100 meg hard disk you have been after!

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Design and Behavior of Concrete Slabs Subject to Blast

by
Tim Knight, PE[1]

Abstract

The protective design field has progressed greatly in recent decades. Guidance on the design and behavior of reinforced concrete slabs subject to blast, in particular, has changed in large part by virtue of the criteria manual updates of the last few years. Significant behavioral considerations have been added, including tension-membrane, soil-structure interaction, and concrete spall from close-in munitions. In addition, the limits on allowable response of conventionally reinforced slabs have risen substantially. Effort is currently underway to continue the improvement of guidance in this field as well as to create new computer program aids for the designer. All of these improvements in guidance will result in higher quality and lower cost protective structures.

Introduction

It is likely that the majority of past structural effort in the field of blast protection has been spent in analysis and design of reinforced concrete wall, roof, and floor slabs. Mass, strength, ductility, and economy combine to make reinforced concrete the structural material of choice for many blast protection applications. These applications cross a wide spectrum of blast protection categories including nuclear, conventional weapon, and accidental explosions. This paper will focus on the nonnuclear categories.

Design criteria as well as guidance on the behavior of reinforced concrete slabs subject to blast have progressed considerably in the past 20 years or so, particularly in the area of nonnuclear blast protection. A great deal of progress was made with the advent of the 1969 accidental explosions triservice manual, TM 5-1300 (Headquarters, Departments of the Army, Navy, and Air Force, 1969). This manual contained significant new guidance on the behavior of reinforced concrete past the elastic range. Blast design guidance in the area of convention weapon attack has progressed in the 1980's and early 1990's with the issuance of Army TM 5-855-1 (Headquarters, Department of the Army, 1986), ESL-TR-87-57 (AFESC 1989), and ETL 1110-9-7 (Headquarters, Department of the Army, 1990) which supplements TM 5-855-1. In addition a 1991 version of the accidental explosions triservice manual (Headquarters, Departments of the Army, Navy, and Air Force, in preparation) is currently being published. Together, these recent manuals have greatly expanded the guidance on the behavior of reinforced concrete slabs subject to blast.

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It is the intent of this paper to discuss a few of the more significant improvements the referenced criteria manuals have made in the design guidance for concrete slabs subject to blast, to bring out a few of the current gaps in the guidance, and to discuss some of the ongoing improvement efforts underway at the Protective Design Center (PDC), US Army Engineer District, Omaha. The improvement efforts underway at the PDC include not only changes to criteria but also computer program development to aid the engineer in more accurately analyzing blast effects. These efforts are a part of the overall continuing commitment by the US Army to improve quality in the protective design field.

Past Criteria and Methods

Prior to the criteria manual upgrades of the last few years, the most extensive and widely used guidance on the design and behavior of reinforced concrete slabs subject to blast was undoubtedly the 1969 version of TM 5-1300. The dynamic behavioral modeling technique presented in this manual is single-degree-of-freedom (SDOF) modeling of flexure, illustrated in Figure 1. Elasto-plastic spring resistance is used in this method to model the formation of hinges at maximum moment points and the subsequent plastic behavior of the member. Response limits for reinforced concrete members are defined in terms of support rotation. For conventionally reinforced members (nonlaced) support rotation is limited to a maximum of 2 degrees. Higher support rotations are allowable in members reinforced with lacing, a system of continuous shear reinforcing zig-zagging throughout the member. However, due to the high cost of this method of reinforcing, it has been generally avoided.

For reinforced concrete slabs subject to blast, a few important behavioral considerations were not covered in past criteria documents. Among these are tension-membrane behavior, close-in spall damage, and soil-structure interaction during ground shock. These considerations will be discussed later.

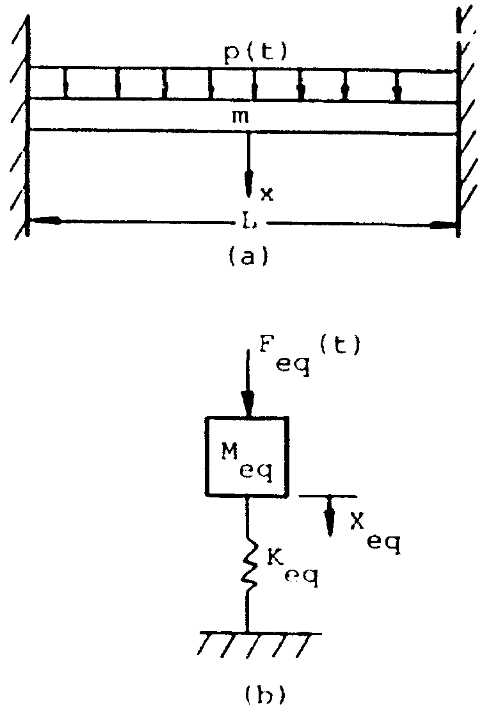

Figure 1. Equivalent single-degree-of-freedom (SDOF) system

Current Criteria and Methods

Response limits

The criteria manual upgrades of the last few years have brought significant change to the guidance on the design and behavior of reinforced concrete slabs subject to blast. The three recent manuals of applicability are the conventional weapons protective design manuals, Army TM 5-855-1 and Air Force ESL-TR-87-57, and the triservice accidental explosions design manual, Army TM 5-1300 (revision in preparation). Also, applicable is Army ETL-1110-9-7 which supplements TM 5-855-1. Since a great deal more test data have been brought to bear in these manuals, some of the past guidance gaps have been closed and some of the past conservatism, caused by uncertainty, has been reduced. SDOF modeling of flexure, without membrane behavior, is still the basic method of dynamic analysis rec-

ommended in these manuals. However, the allowable member response has been increased in all conventionally reinforced (nonlaced) slabs, particularly with the new recognition in these manuals of the large deflections associated with tension-membrane behavior. The previous support rotation limit of 2 degrees has been increased to from 4 to 6 degrees for slabs which respond in flexure only and to from 8 to 20 degrees for slabs which can develop tension-membrane behavior. These limits vary between manuals and are delineated in Table 1.

Table 1
Current Criteria Comparison Reinforced Concrete Slab Flexural Response Limits for Blast[1]

Criteria Source	Support Rotation Response Limit (degrees)	
	Laterally Unrestrained	Laterally Restrained (Tension-Membrane)
Triservice Explosive Safety Design Manual TM 5-1300 (1991)	4	8
Air Force Protective Design Manual ESL-TR-87-57 (1989)	4	12
Army Protective Design Manual TR 5-855-1 (ETL 1110-9-7 (1990))	6	12 moderate damage 20 heavy damage

[1] Based on conventional reinforcing (nonlaced) with stirrups.

The net result of these response limit changes is undoubtedly significantly less costly designs, particularly in the area of protective design for conventional weapons attack.

Tension-membrane behavior

As illustrated in Figure 2, tension-membrane behavior and generally less significant compression-membrane behavior are observed in slabs where the lateral restraint provided by the supporting members is sufficient to develop the membrane forces. These support conditions are not uncommon in blast designed structures and, in fact, have been observed in many blast-tested structures (Woodson 1990). A general discussion of the reinforcing and support condition requirements for tension-membrane behavior can be found in ETL 1110-9-7. As is evident from the previously mentioned response

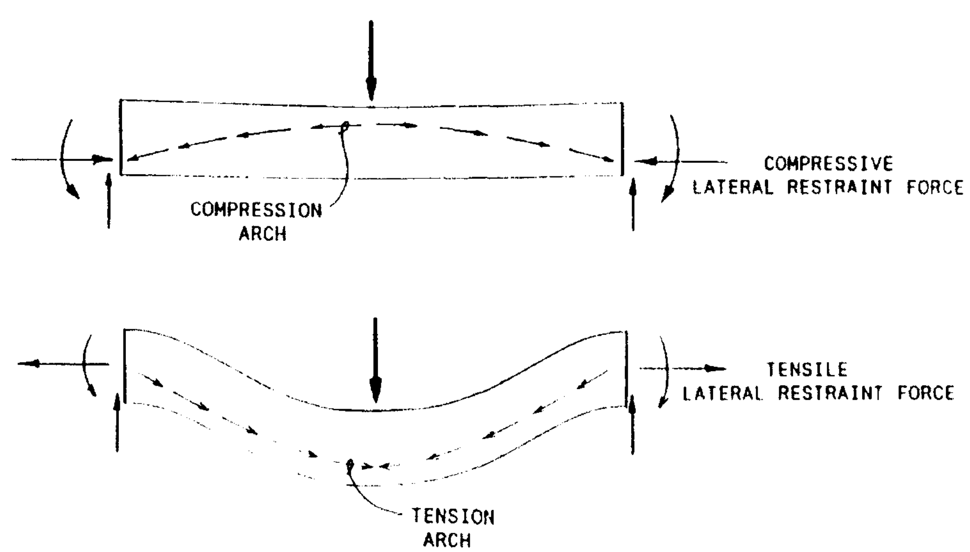

Figure 2. Compression/tension membrane behavior

limit criteria, tension-membrane action can allow a slab to deflect well beyond the limit for flexural action. This is illustrated graphically in Figure 3 and shown in a test specimen in Figure 4.

Although recognized by virtue of the increased allowable deflections, tension-membrane behavior is still not fully accounted for in the current criteria manuals. This is due to the lack of any guidance for determining the actual resistance function such a slab exhibits. Figure 5, taken from TM-5-1300 (revision in preparation), illustrates the current guidance of accounting for tension-membrane behavior by simply extending the elasto-plastic flexural resistance function out to larger support rotations.

Soil-structure interaction

Soil-structure interaction (SSI) is another area where new guidance has been given. The protective design manual, ESL-TR-87-57 (Headquarters, Department of the Air Force, 1989), contains a section covering ground shock interface stress analysis that gives the basic equations and application for SSI. In the context of blast analysis, soil-structure interaction, sometimes more generally termed structure-media interaction, is a procedure whereby the effect of the motion of the structure is accounted for in the ground shock loading function. This is accomplished by recognizing that the soil-structure interface pressure is directly related to the relative velocity of the soil and the structure and that very early in the loading history this relative velocity can decrease dramatically, resulting in a similar decrease in the interface pressure. The equation of motion for the SSI model is shown in Figure 6. Figure 7 is a comparison of measured test data with that calculated using this model. The comparison illustrates both the agreement of the measured and calculated interface stresses and the significant reduction in stress, even below the incident free-field value, that the SSI model shows. The inclusion of the SSI phenomenon in the blast design guidance is a very significant step forward because it can result in much more economical below grade structures.

Spall damage

In the area of concrete spall damage from close-in munitions blast the recent manual updates have added significant new guidance. High-velocity concrete spall can emerge from the protected side of a slab by virtue of the action of high-pressure shock waves or high-velocity fragments impacting the exposed side. As illustrated in Figure 8, this effect can be particularly pronounced for close-in cased charges in air where the high-pressure airblast acts in combination with a relatively dense fragment distribution. The 1991 triservice accidental explosion design manual offers procedures for predicting the occurrence of spall from airblast alone or from individual fragment impact. The 1989 Air Force protective design manual goes further by presenting empirical spall damage curves for both buried and aboveground concrete structures. The aboveground spall curves are for bare and cased charges with an influence factor to account for the relative weight of the casing. Although somewhat useful for design, the aboveground spall curves are currently limited in their applicability to full-scale structures and in the adequacy of the casing influence factor.

In-Progress Improvements to Criteria and Methods

In the field of blast design guidance for reinforced concrete slabs, great improvement has been realized in recent years. However, changes are occurring rapidly in this field and there are always many areas in which more improvement can be made, both in criteria and in methodology. Currently at the PDC, several such improvement efforts are underway, as follows:

- **Spall Damage Upgrade.** Bring sophisticated analytical study together with the empirical data base to improve the existing damage prediction guidance for

aboveground cased and bare charges. Incorporate into criteria manual.

- **Tension-Membrane Modeling.** Develop tension-membrane resistance-displacement model to improve SDOF analysis of laterally restrained members. Incorporate into criteria manual.

- **BLASTX Upgrade.** Develop interactive preprocessor and postprocessor to facilitate general Corps-wide use of BLASTX PC program in the US Army Engineer Waterways Experiment Station (WES), Information Technology Laboratory (ITL), Library. BLASTX computes air shock and gas pressure effects within a room or interconnected rooms due to internal or external explosion(s).

- **Develop Ground Shock Program.** Develop interactive PC based computer program with pre- and postprocessor, to compute ground shock effects on any side of a buried structure. Reflections and reliefs off any planar layer will be accounted for. Free-field stress and velocity histories will be output for use in soil-structure interaction analysis. Incorporate into WES ITL Library.

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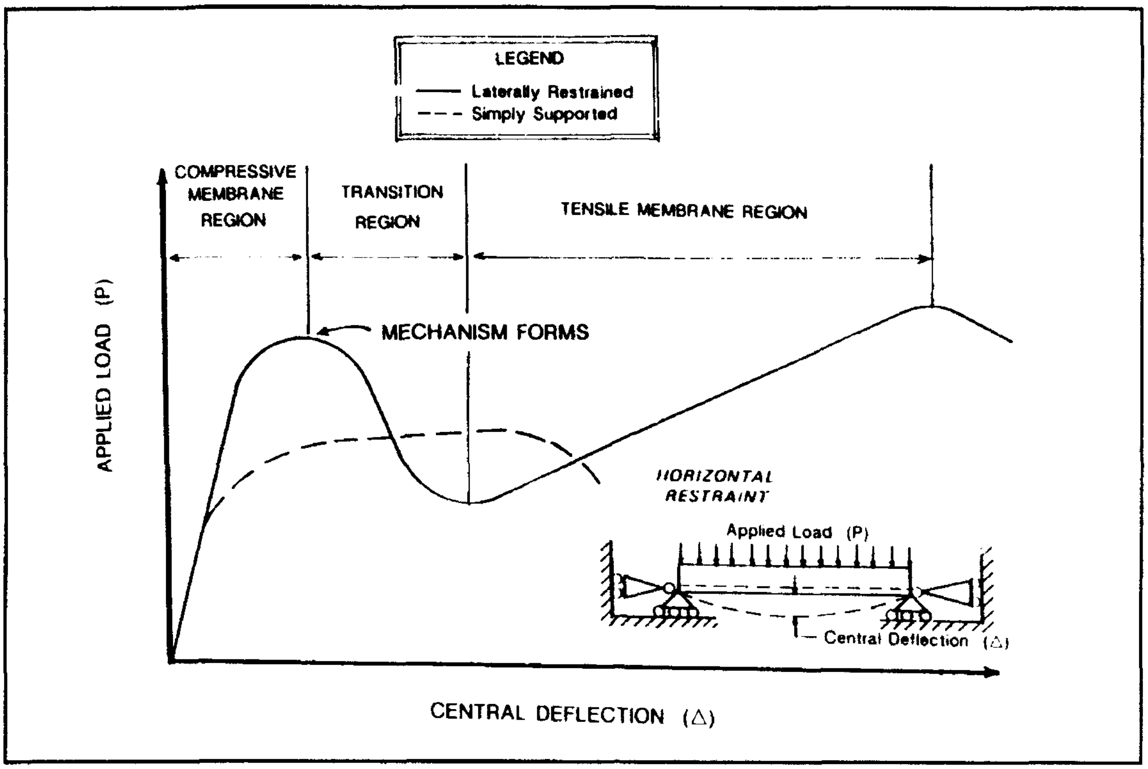

Figure 3. Idealized load-deflection curve for a laterally restrained slab

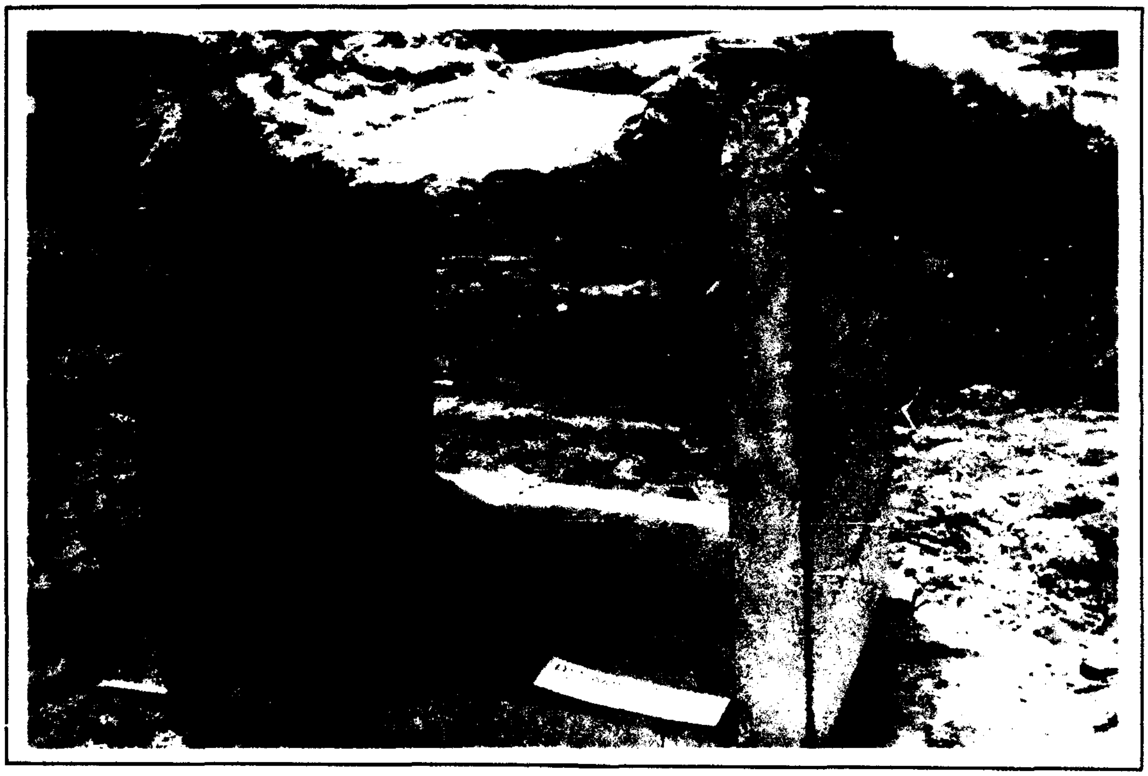

Figure 4. Test specimen showing tension-membrane behavior (23° support rotation)

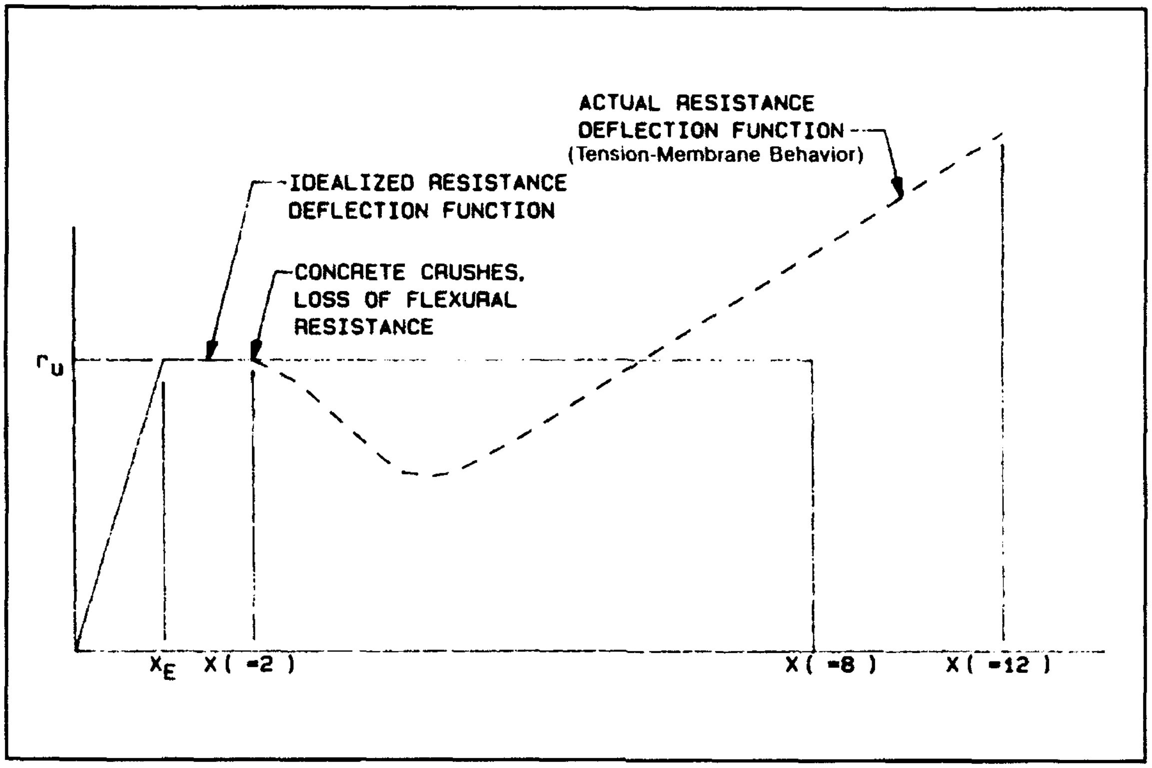

Figure 5. Idealized resistance-deflection curve for large deflections (from 1991 TM 5-1300)

The Soil-Structure Interaction (SSI) model is based upon a simple formulation of a plane wave reflecting from a rigid deforming mass. The basic equations are:

$$\sigma_i = \sigma_{ff} + \rho c\,(V_{ff} - \dot{x}) = m\ddot{x} + R$$

where

σ_i = interface stress

σ_{ff} = incident free-field stress

V_{ff} = incident free-field particle velocity

$\dot{x}$ = structure velocity

$\ddot{x}$ = structure acceleration

R = structure resistance

m = structure mass per unit area

ρc = characteristic impedance of the soil

Figure 6. Soil-structure interaction (SSI) model

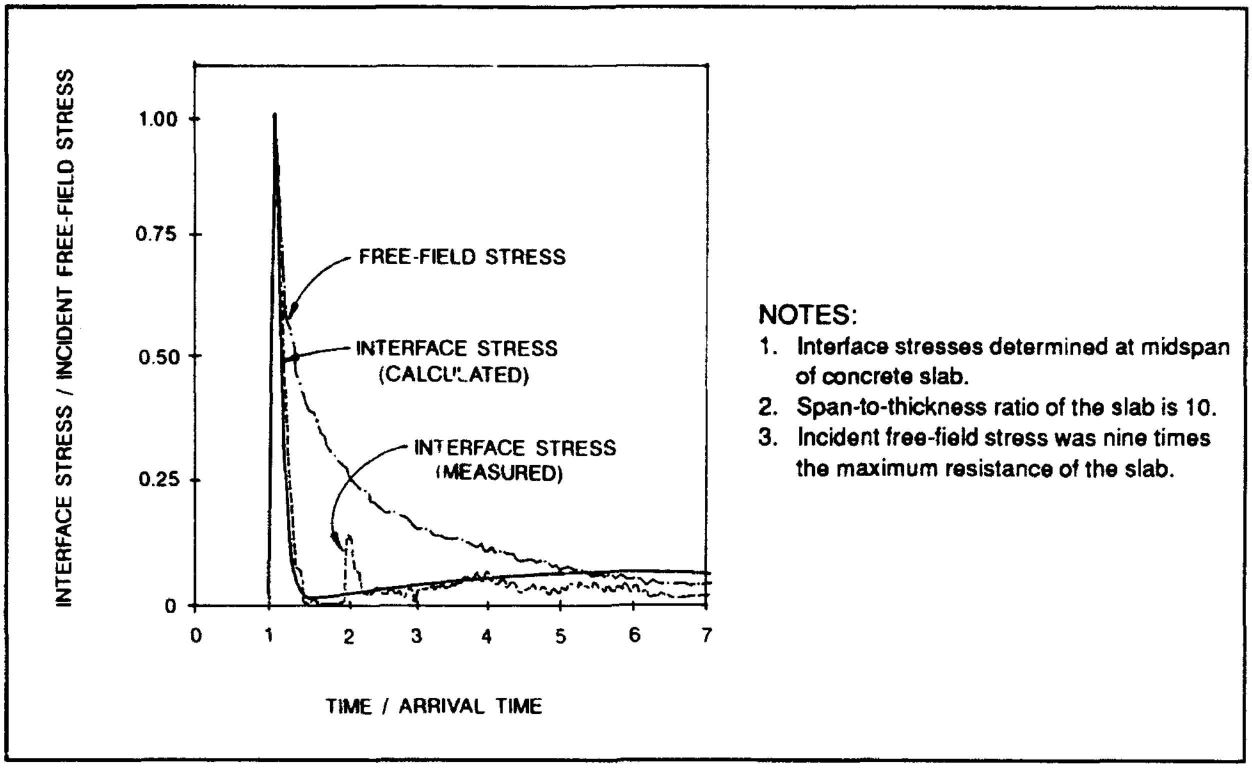

Figure 7. Soil-structure interaction (SSI): calculated versus test data

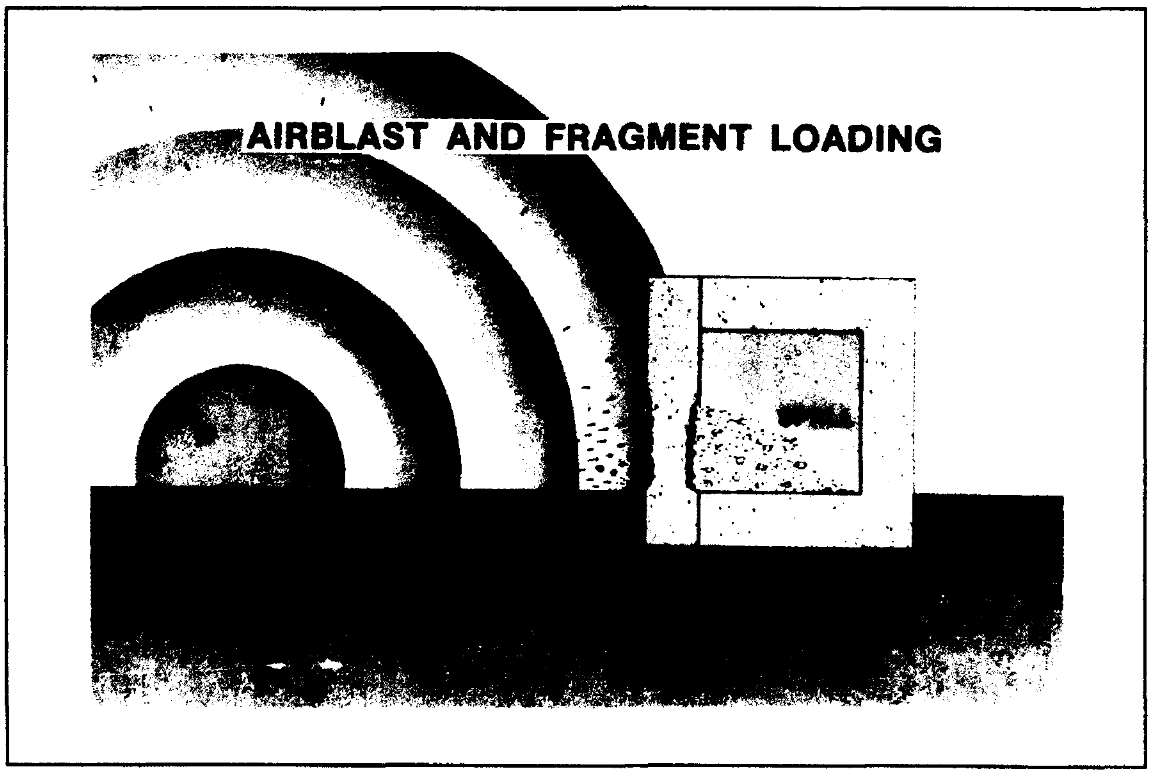

Figure 8. Spalling caused by airblast and fragment loading

Parameters Affecting the Response of Slabs to Conventional Weapons Effects

by
Stanley C. Woodson, PE[1]

Abstract

Several parameters play key roles in enhancing the ductility of a blast-resistant reinforced concrete slab subjected to conventional weapon effects. Data for 258 tests were collected. These tests consisted of static and dynamic loadings of reinforced concrete slabs and box-type structures having either lacing bars or stirrups or no shear reinforcement. The data base is presented in a technical report recently published at the US Army Engineer Waterways Experiment Station.

A study of the data base indicates that the primary parameters affecting the large deflection behavior of reinforced concrete slabs include support conditions, amount and spacing of principal reinforcement, scaled range, and span-to-effective-depth ratio. An understanding of how these parameters interact to enhance the ductility of a slab will lead to the design of more economical structures. This study contributed significantly to the develop-ment of new shear reinforcement design criteria and associated response limits for protective structures designed to resist the effects of conventional weapons. The new criteria were recently published as an Engineer Technical Letter (ETL). The purpose of the ETL is to improve constructibility and economy associated with protective construction while reflecting an improved understanding of the physical parameters involved in the dynamic response of a reinforced concrete slab.

Introduction

A literature search was conducted to gather available test data of reinforced concrete slabs loaded to failure or to large deflections (statically and dynamically). A primary objective of the study was to determine how shear reinforcement details interact with other physical details to affect the response limits of a slab. Shear reinforcement used in blast-resistant structures usually consists of either lacing bars or single-leg stirrups (Figure 1). Lacing bars are reinforcing bars that extend in the direction parallel to the principal reinforcement and are bent into a diagonal pattern between mats of principal reinforcement. The lacing bars enclose the transverse reinforcing bars which are placed outside the principal reinforcement. The cost of using lacing reinforcement is considerably greater than that of using single-leg stirrups due to the more complicated fabrication and installation procedures.

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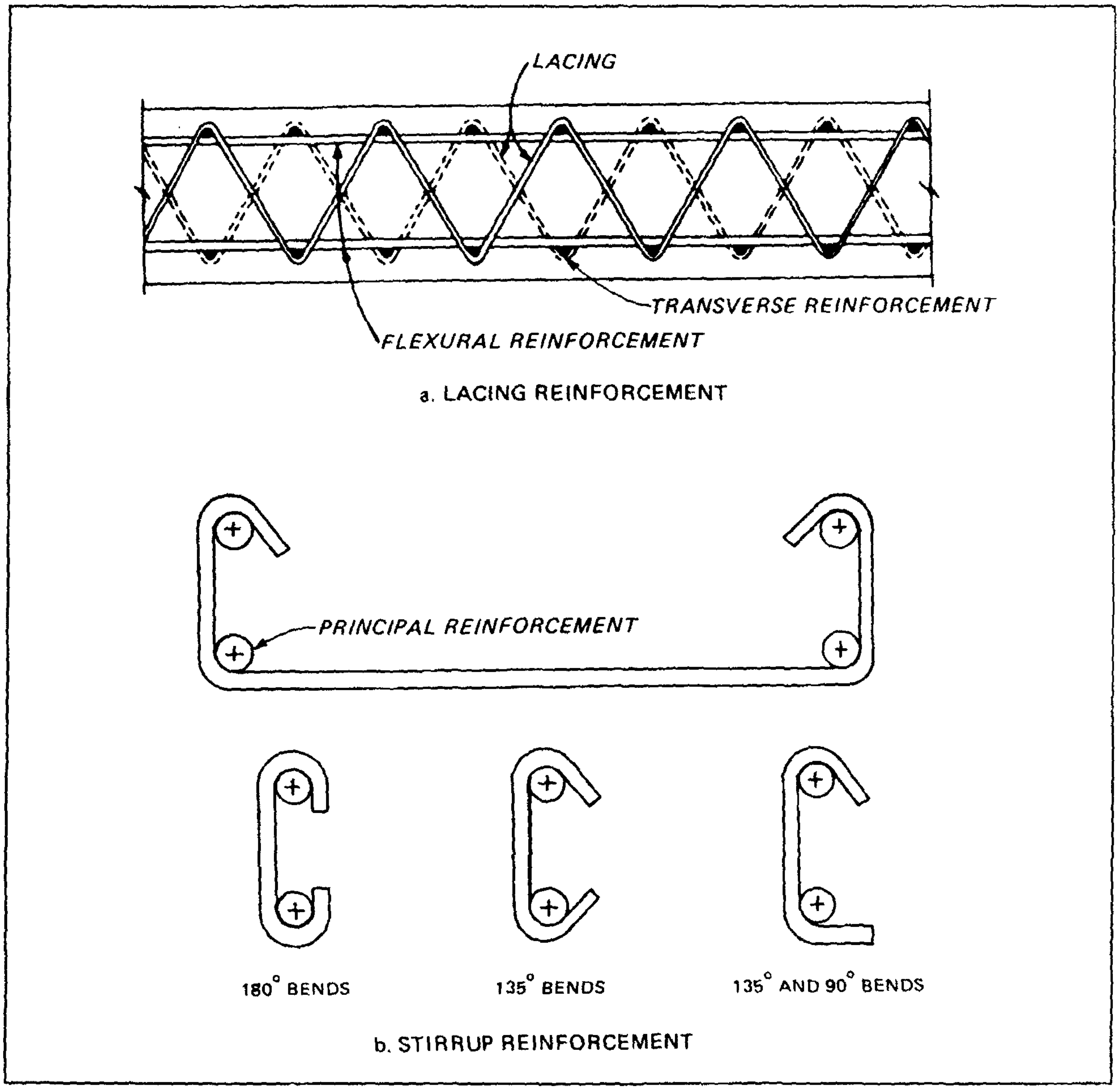

Figure 1. Shear reinforcement

As will be discussed, most design guides and manuals for blast-resistant reinforced concrete structures stipulate the use of shear reinforcement irrespective of shear stress levels. Allowable response limits are often tied directly to principal reinforcement and shear reinforcement details with little regard to the affects of other structural details. The data base developed during this literature search allowed the study of the relative significance of many structural parameters that affect the response of slabs.

Design Manuals

The Tri-Service Manual

The Tri-Service Manual (Headquarters, Departments of the Army, the Navy, and the Air Force 1969) is the most widely used manual for structural design to resist blast effects. The manual's Army designation is TM 5-1300. A recently completed revision of TM 5-1300 is available in draft form, and criteria from Volume IV of the draft (Dede and Dobbs 1987) will be briefly discussed.

In general, the draft manual states that the compression concrete of a reinforced concrete element crushes at a deflection corresponding to 2-deg support rotation. This crushing of concrete is assumed to be "failure" for elements without shear reinforcement. Section 4-9.1 of the manual states that single-leg stirrups will restrain the compression reinforcement for a short time into its strain-hardening region until failure of the element occurs at a support rotation of 4 deg. It further states that lacing reinforcement will restrain the flexural reinforcement through its entire strain-hardening region until tension failure of the principal reinforcement occurs at a support rotation of 12 deg. Therefore, the manual requires the use of laced reinforcement when an element is designed to undergo support rotations between 4 and 12 deg. The exception to this is when support conditions provide enough lateral restraint that in-plane tensile membrane forces may be developed. In this case, an element with stirrups is considered to be capable of sustaining a support rotation of 8 deg.

TM 5-1300 is intended for the design of structures, including nonmilitary structures, to resist the effects of accidental explosions. Therefore, it provides a conservative design with a high degree of safety. However, its criteria have been adopted by many organizations, even for military appli-cations. This has resulted in what may be excessive use of expensive laced reinforcement, as well as the construction of massive members due to conservative limitations on allowable support rotations.

Army Technical Manual 5-855-1

TM 5-855-1 (Headquarters, Department of the Army 1986) is intended for use in the design of hardened facilities to resist the effects of conventional weapons. Chapter 9 of TM 5-855-1 discusses the design of shear reinforcement. The criteria presented are primarily based on the guidance of ACI 318-83 (ACI 1983) with consideration of available test data. The maximum allowable shear stress to be contributed by the concrete and the shear reinforcement is given as $11.5(f'_c)^{1/2}$ for design purposes as compared to a value of $8(f'_c)^{1/2}$ given by ACI 318-83. An upper bound value for the shear capacity of members with web reinforcing is given as that corresponding to a 100 percent increase in the total shear capacity outlined by ACI 318-83 and consisting of contributions from the concrete and shear reinforcement. An important statement concerning shear reinforcement in one-way slabs and beams is given in Section 9-7 which reads as follows.

> Some vertical web reinforcing should be provided for all flexural members subjected to blast loads. A minimum of 50-psi shear stress capacity should be provided by shear steel in the form of stirrups. In those cases where analysis indicates a requirement of vertical shear reinforcing, it should be provided in the form of stirrups.

TM 5-855-1 states that shear failures are unlikely in normally constructed two-way slabs, but the possibility of shear failure increases in some protec-tive construction applications due to high-intensity loads. Shear is given as the governing mode of failure for deep, square, two-way slabs. In the event shear capacity is required above that provided by the concrete alone, additional strength can be provided in the form of vertical and/or horizontal web reinforcing. For beams, one-way slabs, and two-way slabs, the manual recommends a design ductility ratio of 5.0 to 10.0 for flexural design.

Engineer Technical Letter 1110-9-7

ETL 1110-9-7 (Headquarters, Department of the Army 1990) is partly the result of a literature search and parameter study conducted by Woodson (1990). The ETL is considered to be a supplement to TM 5-855-1 in that it provides more in-depth guidance on the design of reinforced concrete slabs to resist the effects of conventional weapons. The ETL goes beyond the conservatism of TM 5-1300 by recognizing the mission of military structures. Yet, the allowable response limits are based on existing data and are dependent upon the proper incorporation of several key

construction parameters. This consideration of several parameters in defining allowable support rotations sets the ETL apart from most design guidance for blast-resistant structures. This is particularly true for the guidance given in TM 5-1300 which is very dependent on the type of shear reinforcement used (stirrups versus lacing).

Parameter study

The data base is presented and discussed in detail in the Waterways Experiment Station Technical Report written by Woodson (1990). Due to space limitations the following discussion of the study will emphasize the primary findings that have impacted the Corps' design guidance.

The parameters generally available from reports and papers discussing test data include shear reinforcement details, amount and spacing of principal reinforcement, support conditions, scaled range for explosive testing, and span-to-effective-depth (L/d) ratio. The amount of principal reinforcement is usually given by the tension reinforcement ratio (ρ). The scaled range (z) refers to the size and standoff of the explosive charge weight and is expressed in the units of $ft/lb^{1/3}$.

Generally, roof, floor, and wall slabs of a reinforced concrete protective structure are laterally restrained. Lateral restraint is an important parameter in that it must exist for tension membrane forces to develop. These membrane forces develop when a slab undergoes large deflections and the slab takes the shape of a catenary. In this case, the slab must be sufficiently anchored into the support members, and the support members must have sufficient strength to resist the applied forces.

Most of the available data for test slabs indicate that the slabs were laterally restrained. These slabs include components of box-type structures as well as slabs that were laterally restrained in test devices or reaction structures.

Most of the data for laterally restrained slabs (as opposed to box-type structures) tested in reaction structures are the results of tests conducted in the 1960's during the development of the original TM 5-1300. This group of data primarily pertained to either laced slabs or slabs with no shear reinforcement. It appears that slabs containing stirrups were generally placed in the same category as slabs having no shear reinforcement. Hence, TM 5-1300 is more restrictive for slabs containing stirrups rather than lacing bars. This is also true for the new draft TM 5-1300. Through the study of more recent data from tests on slabs (including box-type structures) containing stirrups, a better understanding of the role of shear reinforcement in enhancing the ductility of a blast-resistant slab has been achieved and incorporated into the new ETL.

For laterally restrained slabs, the L/d ratio plays a significant role in the response of a slab. Generally, slabs are considered to be "deep slabs" when the L/d ratio is below a value of 5 or 6. It is well known within the structural engineering profession, primarily from numerous beam tests, that deep members tend to respond in a more brittle (shear-type) failure mode as compared to the more ductile response of normally proportioned members. From the data base, it is observed that slabs with low values of L/d that were tested at $z = 1.0\ ft/lb^{1/3}$ incurred only slight damage. For these slabs, support rotations were low (5 to 7 deg) even when no shear reinforcement was used. Generally, wall slabs of buried boxes having L/d values of 10 to 15 experienced large support rotations (15 to 29 deg) and were damaged to near incipient collapse. However, a wall slab that had a L/d of 7 and was tested at $z = 0.75\ ft/lb^{1/3}$ sustained a support rotation of 26 deg without breaching, although there was no shear reinforcement. Support rotations are not available for the 1960's test data, but rather, the damage was generally reported as being light, moderate, or heavy.

A review of data for the laterally restrained laced slabs tested at $z < 2.0\ ft/lb^{1/3}$ provides some insight into the difference in the behavior of laced and nonlaced slabs. The fact that both a laced slab and a slab with no shear

reinforcement incurred heavy damage when tested at $z = 1.5$ ft/lb$^{1/3}$ and 1.25 ft/lb$^{1/3}$, respectively, somewhat questions the significance of lacing. When laterally restrained laced slabs with $\rho = 2.7$ percent were subjected to low z values of 0.3 and 0.5 ft/lb$^{1/3}$, they experienced heavy damage and partial destruction, respectively. It is interesting to note that a laterally unrestrained slab with no shear reinforcement and $\rho = 2.7$ incurred only medium damage at $z = 0.5$ ft/lb$^{1/3}$. This indicates that the effects of the large ρ of 2.7 percent overshadowed the effects of shear reinforcement on the response of these slabs. Hence, the effects of the principal reinforcement details must be considered in the development of design criteria. Proper consideration of this parameter will greatly relax shear reinforcement requirements for large-deflection behavior.

Data for laterally unrestrained, nonlaced slabs tested at $z < 2.0$ ft/lb$^{1/3}$ are very limited. It is obvious from the data base that unrestrained slabs with low percentages of tension reinforcement are susceptible to experiencing major damage when $z < 2.0$ ft/lb$^{1/3}$. For example, damage levels ranged from slightly damage to total destruction for slabs that had an L/d value of 10, a ρ of 0.15 percent, and were tested at z values from 1.7 to 1.0 ft/lb$^{1/3}$. Even for large values of ρ, both laced and nonlaced slabs incurred significant damage. For example, an unrestrained slab with no shear reinforcement, $\rho = 2.7$ percent, and $L/d = 7$ incurred medium damage at $z = 0.5$ ft/lb$^{1/3}$. An unrestrained laced slab with $\rho = 2.7$ percent experienced heavy damage at $z = 0.5$ ft/lb$^{1/3}$. Again, it appears that parameters (in this case, support conditions and tension reinforcement) other than shear reinforcement were more important than shear reinforcement details.

Design Guidance

This study supports the development of the new shear reinforcement design criteria and associated response limits given in ETL 1110-9-7 for protective structures designed to resist the effects of conventional weapons. The recommended response limits that were incorporated into the ETL are given in Table 1.

Table 1
Recommended Response Limits for Reinforced Concrete Slabs

Lateral Restraint Condition	Damage Level	Response Limit deg
Unrestrained	—	6
Restrained	Moderate	12
Restrained	Heavy	20

As previously discussed, laterally unrestrained and laterally restrained slabs behave differently because tension membrane forces can develop in a one-way slab only if the slab is laterally restrained at the supports. However, lateral restraint is inherent to two-way slabs. Table 1 presents allowable support rotations for laterally restrained slabs based on acceptable damage levels that must be chosen by the designer, depending on the purpose of the structure. Moderate damage means that significant concrete scabbing and reinforcement rupture have not occurred and the dust and debris environment on the protected side of the slab is moderate; however, large slab motions will occur. Such a damage level may be acceptable for the protection of personnel and sensitive equipment. Heavy damage means that the slab is at incipient failure, and significant reinforcement rupture may have occurred over much of the slab. In this case, the slab may resemble a reinforcing grid suspending concrete rubble.

Based on the limits of the data base, the response limits given in Table 1 should only be used if (1) the scaled range exceeds 0.5 ft/lb$^{1/3}$, (2) the clear span to effective depth ratio (L/d) exceeds 5, (3) the principal reinforcement spacing is minimized (never exceeding the effective depth of the slab), and (4) adequate stirrups are provided. Stirrup reinforcement is required to provide adequate concrete confinement and principal steel support in the large-deflection region. Stirrups should be required along each principal reinforcing bar at a maximum spacing of d/2 when $z < 2$ ft/lb$^{1/3}$,

and at a maximum spacing of d at larger scaled ranges. When stirrups are required to resist shear, stirrup spacing should not exceed d/2. In accordance with TM 5-855-1, all stirrup reinforcement should provide a minimum of 50 psi shear stress capacity. Single-leg stirrups having a 135-deg bend on one end and at least a 90-deg bend on the other end are recommended for economy.

It is observed from the data base that flexible slabs that are laterally restrained are much less likely to fail in direct shear because early in the response, lateral compression membrane forces will act to increase the shear capacity, and later in the response, shear forces tend to be resolved into the principal reinforcement during tension membrane action. Tests indicate that direct shear failure can occur in slabs subjected to impulsive loads. It is generally known that a shear-type failure is more likely to occur in reinforced concrete members with small L/d values than it is in those with large L/d values. Since the data base indicates that laterally restrained slabs with $L/d \geq 8$ are unlikely to experience direct shear failures, consideration for the design of details to resist direct shear is only recommended for laterally restrained slabs having $L/d < 8$ and for all laterally unrestrained slabs. This is considered to be conservative, but the degree of conservatism is unknown due to gaps in the data base.

Conclusions

Tests conducted in the past few years indicate that slabs with stirrups can sustain large support rotations and that several slab parameters contribute significantly to ductile behavior.

New guidelines have been developed for response limits and shear reinforcement details for the design of structures to resist the effects of conventional weapons. However, additional data are needed to further improve the accuracy and reduce the conservatism of the design criteria.

Acknowledgements

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Corps Masonry/Quality Assurance

by
Ervell A. Staab, PE[1]

Abstract

Masonry continues to be the primary building product used at military installations throughout the United States. High quality, economical structures of reinforced masonry are being built. There has been an increasing effort within the Corps the past several years to assure high quality masonry products. This has involved revising the Masonry Manual (Departments of the Army, Navy, and Air Force 1982 (Feb, Aug)), updating the Guide Specifications, developing masonry details, developing a masonry course, initiating masonry research, organizing a Masonry Task Group, and participating on national masonry committees. The Corps is taking some bold steps in requiring reinforced masonry design, requiring minimum reinforcement for all structural walls, removing arbitrary height limitations, limiting maximum bar sizes, and promoting open-end units. The Guide Specifications are being updated to reflect recent changes in masonry products and construction. Masonry details are being developed that will result in improved constructibility and performance. The research initiatives will result in increased productivity through lighter weight units and greater assurance of performance through proper development and splice lengths for reinforcement. The Masonry Task Group is actively involved in developing computer programs and is providing guidance and oversight on all masonry activities. Participation on national masonry committees is providing a broad overview on all codes and standards, including the development of the new limit states design method. The masonry course provides a means for technology transfer and assures improved design and detailing practices. This combined effort will continue to assure quality masonry products for our customers that are economical and will provide the serviceability needed for low maintenance facilities.

Introduction

Masonry is a field assembled product that requires the coordinated efforts of the designer, product suppliers, contractor, and the construction quality assurance staff. All participants play a vital role in the process. There has been considerable effort recently within the Corps to assure quality masonry products through revising the Masonry Manual (Departments of the Army, Navy, and Air Force 1982 (Feb, Aug)), updating the Guide Specifications, developing masonry details, developing a masonry course, initiating masonry research, forming a Masonry Task Group, and participating on national masonry

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committees. The combined effect will result in more efficient, economical, constructible masonry with improved serviceability. The Corps needs to be in a leadership role to effect industry changes for the national interest. This paper addresses primarily the design engineering aspects of recent Corps activities to enhance quality masonry products.

Masonry Manual

The revised Masonry Manual (Departments of the Army, Navy, and Air Force 1982 (Feb, Aug)) contains a number of significant changes. These changes will enhance the quality of the masonry product without complicating the design. The design is based on the working stress method and is essentially unchanged from previous Corps manuals. It is coordinated with the Seismic Manual. All masonry design criteria and procedures are in the revised Masonry Manual. The revised Seismic Manual contains only the specific seismic requirements, such as; minimum reinforcement requirements for various seismic zones, minimum connections, and special detailing practices.

Reinforced masonry

All masonry in seismic and nonseismic areas will be designed as reinforced masonry except that partitions in nonseismic areas may be designed as unreinforced masonry when satisfying the unreinforced masonry requirements of American Concrete Institute (ACI) 530, "Building Code Requirements for Masonry Structures" (ACI 1988). The revised manual does not contain design requirements for unreinforced masonry. Unreinforced masonry performs very poorly in high winds and during major seismic events, usually leading to structural failures or collapse. All parts of the country are exposed to either high wind or high seismic effects. Note also that tension in mortar beds is not permitted for resisting uplift loads due to wind or seismic effects, thereby, requiring at least some vertical reinforcement for those conditions. Life safety is the primary responsibility of the structural engineer. Designing unreinforced masonry in high wind areas and in high seismic areas is not responsible engineering. Designing reinforced masonry is quality engineering.

Minimum reinforcement

All exterior, bearing and shear walls (structural walls) are required to have minimum reinforcement. One vertical reinforcing bar is required continuously from support to support at each corner, at each side of each opening, at each side of control joints, at ends of walls, and at a maximum spacing of 6 ft. This minimum reinforcement will be the same size as the vertical reinforcement provided for flexural stresses. These requirements are the same as in the current seismic manual for Zone 1 exception, except the maximum spacing is reduced from 8 ft to 6 ft. The reduced spacing will provide a higher assurance of ductile behavior. Horizontal reinforcement is required continuously at floor and roof levels and at the tops of walls. Horizontal reinforcement will also be provided above and below openings. Horizontal reinforcement for stacked bond will be at least 0.0007 times the vertical cross-sectional area placed in bed joints or in bond beams spaced not more than 48 in. apart. The minimum reinforcement will provide for some ductility during unusual loading and will enhance performance for environmental effects. Nonstructural partitions need only the reinforcement required for crack control. Control joint spacing is based on the horizontal joint reinforcement provided and reinforcement around openings to control cracking. See Plates 1, 2, and 3 for minimum reinforcement requirements.

Maximum bar size

Large bars in masonry cells have several major drawbacks. It is usually not possible to fully develop large bars in flexure, because the compression in masonry normally controls resulting in low tensile stresses in the steel which is generally not economical. The masonry cells containing the reinforcing bars are usually not large enough, or are not aligned, to allow for proper placement and grouting. In addition, allowance for splicing is usually required. Constructibility can also

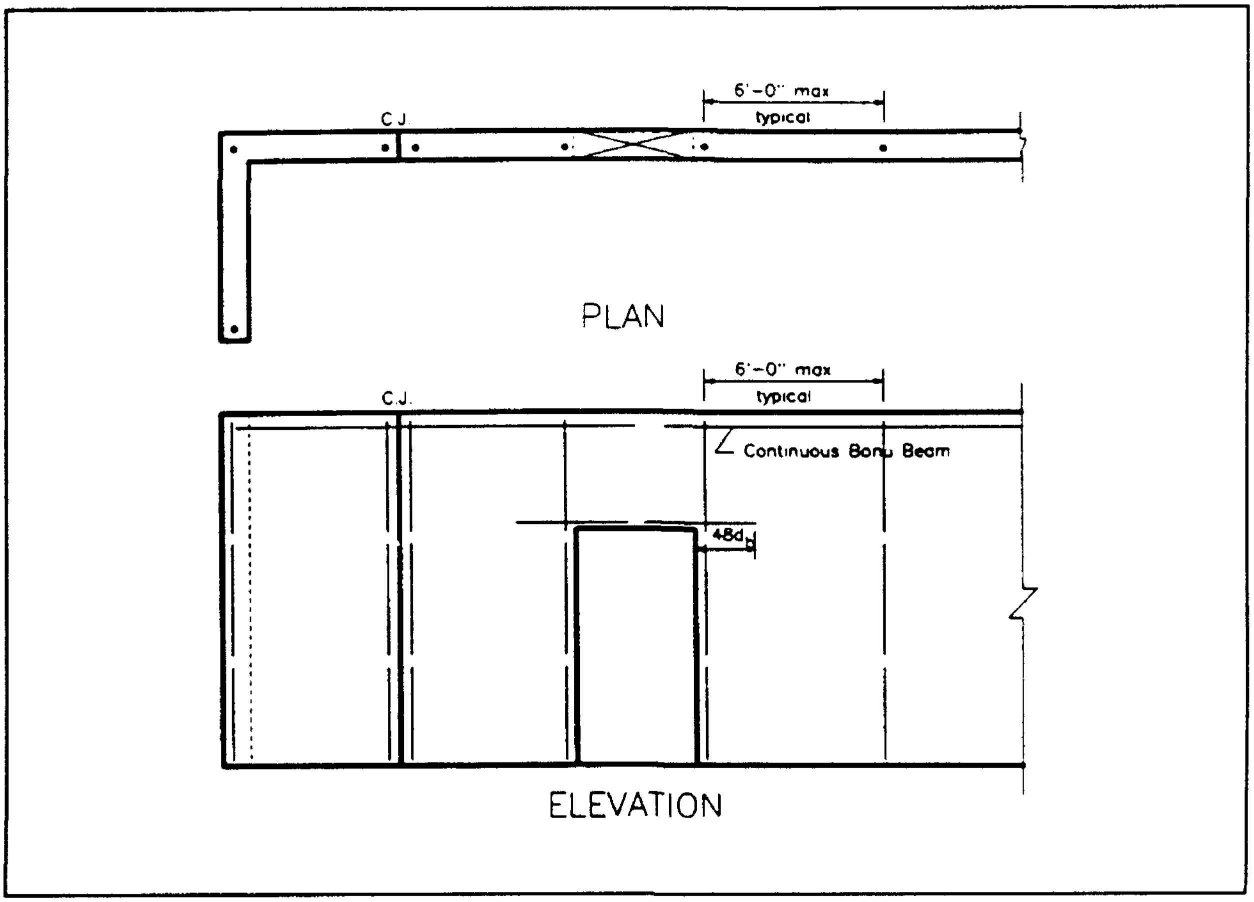

Figure 1. Running bond minimum reinforcement

be a problem due to the long splice lengths. Unless open-end masonry units are used, they may have to be lifted high over the projecting steel. The preferred reinforcing bar sizes are Nos. 4, 5, and 6. The reinforcing should not exceed No. 6 bars in 6-in. units, No. 7 bars in 8-in. units, and No. 8 bars in 10-in. and 12-in. units. These requirements result in reasonable steel ratios and splice lengths and better distribution of reinforcement. Grouting continues to be a constructibility problem. Limiting the bar sizes will improve masonry construction.

Splice lengths

Splice lengths are a major concern in that there is a wide range of splice length requirements by the various codes. Splice lengths can vary from as little as 40 bar diameters to as much as 72 bar diameters and more. The revised manual recommends 48 bar diameters or ACI 530.88 (ACI 1988) requirements.

This is an area that needs continued attention until more research is completed to validate current practices. Reducing the number of splices should be the goal of every design. This can be accomplished through careful detailing and the use of open-end units, both leading to higher quality masonry. See Figure 2 for splice lengths by various codes.

Arbitrary limits

The height/thickness (h/t) limitations as commonly used in past manuals are no longer required in either seismic or nonseismic areas. Instead, designs based on established principles of mechanics, within the allowable stresses and limits, are acceptable. Generally h/t values exceeding 24 should consider P-Delta effects. This will generally allow for higher walls than in the past for the same thickness. Reinforced masonry design, minimum reinforcement, limited bar sizes and open-end units all

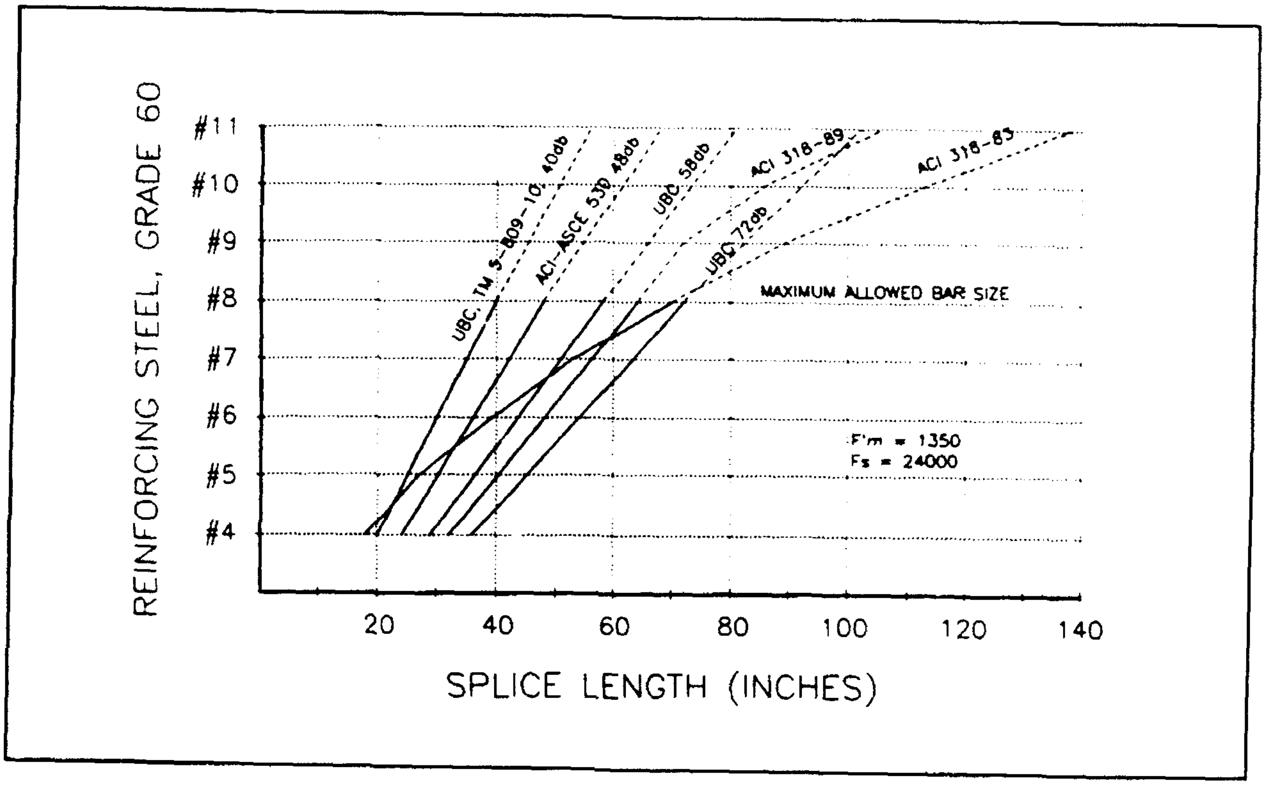

Figure 2. Splice lengths

contribute toward a higher quality wall which allows a relaxation of this previous requirement.

Crack control

Crack control for masonry is similar to past requirements except that the aspect ratio, length to wall height, is included and the spacing near masonry-bonded corners, or similarly restrained conditions, should be reduced to half the normal spacing. Brick expansion joint spacing has been reduced due to an assumed increase in the moisture expansion potential. In addition, brick expansion joints are recommended to be placed approximately 6 to 10 ft from corners or any other offsets that could induce cracking. There have been numerous cases of cracking at corners and at offsets or at brick pilaster projections. Crack control detailing has been severely lacking on many past projects resulting in excessive cracking and often requiring remedial measures. This is a common problem throughout the masonry industry. Brick expansion joints are also required below shelf angles. They need to allow for brick expansion and building shortening due to creep and other effects that could cause the veneer to become a load bearing wall. The guidance in the Masonry Manual has been expanded and crack control is emphasized in the Corps' masonry course, which is leading to improved crack control measures.

Open-end units

The masonry units in general use today were developed prior to the common use of reinforced masonry. The shapes are not conducive to reinforcing and generally lead to poorly reinforced masonry walls. Cells do not align, placement of steel is difficult, and grouting often results in detrimental voids. The use of open-end units should be used at all vertical reinforcement locations. Open-end units, along with 8- and 16-in.-deep lintel units, placed on their side allows for steel placement for almost all conditions with general spacings as low as 16 in. and closer spacing adjacent to corners and openings with the lintel units. The alternative is to use units with cells that align so that cleanouts can be

provided at the base of the wall and high-lift grouting methods can be implemented. Concrete masonry units commonly called formblocks may be used. These are required to be fully grouted when they do not meet American Society for Testing and Materials (ASTM) C 90-90 (ASTM 1990) dimension requirements. It is imperative that the open-end units, the formblock units, or other units that allow for steel placement and grouting be used. This will result in a giant leap forward for reinforced masonry. See Figure 3 for open-end units.

Quality assurance

The revised masonry manual contains specific guidance for engineering support of quality assurance. It covers contract drawings, specific details, shop drawings, repetitive problems, instructions to the field, and site visits. The use of this guidance will eliminate repetitive problems normally encountered in the field. It requires the designer to develop adequate drawings and details for construction which will benefit both the contractor and the quality assurance staff. Shop drawings need to be checked by a structural engineer. Instructions to the field provide guidance to the quality assurance staff covering the most critical features of masonry construction. The site visits are intended to facilitate discussion between construction field office personnel and the designer on special features of the design, to provide feedback on problem areas and design improvements to engineering, and to see that the design intent is being reflected in the construction. Site visits should be made during critical phases of construction and not just in response to isolated field problems. Generally several issues addressed here are lacking on most masonry projects.

Nondestructive evaluation

Existing masonry structures often need to be evaluated to determine their structural integrity for continued use. Occasionally new structures have to be evaluated due to design or construction deficiencies. The revised Masonry Manual contains guidance on techniques currently available that may be used to assist the designer in obtaining information about the qualities of existing structures. Their use normally requires extensive knowledge of their application and considerable judgement in evaluating the results. This is an evolving area in the masonry industry. The manual summarizes the state of the art.

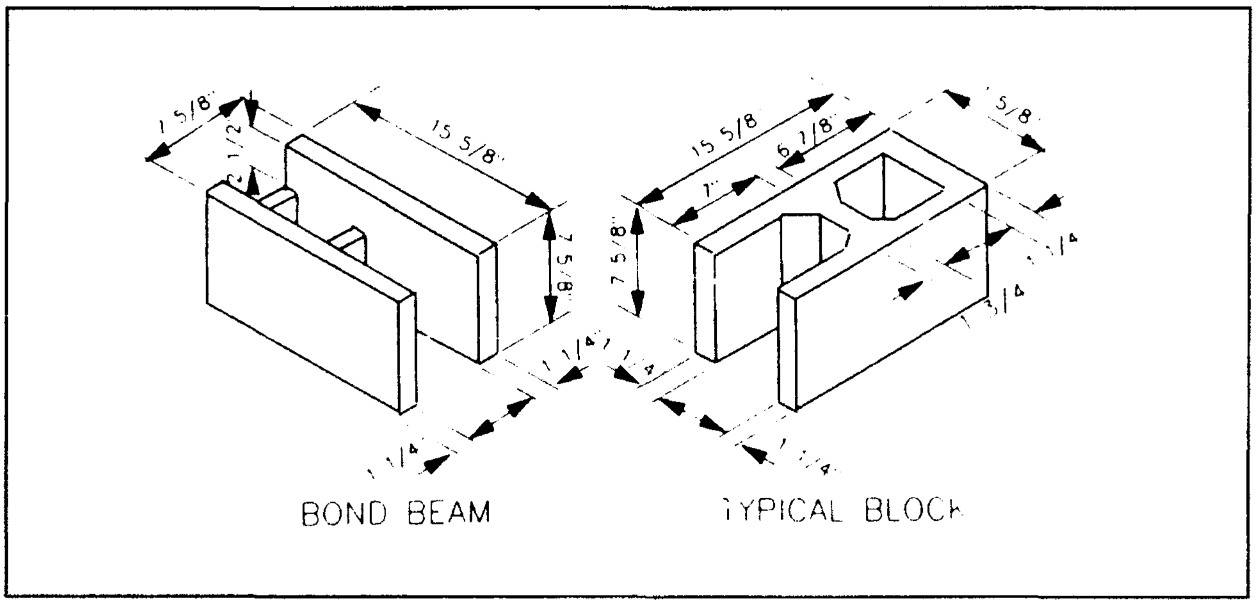

Figure 3. Open-end units

Guide Specifications

The guide specifications are being updated to reflect changes in masonry products and construction practices. There is good cause to take a closer look at the sample wall panels. Making a vertical sawcut through a reinforced grouted cell would add considerably to determining the effectiveness of the grouting procedure. The masonry cement issue needs further study to determine if and where it can be used with a high level of confidence. The high air content of masonry cement is of particular concern. Both designers and contractors are leery of potential problems and often refuse to use it. Various masonry mortars are now available, including premixed mortars, that need to be evaluated for limits of application. Testing various masonry products for efflorescence should be addressed. Testing methods for strength and constituency along with frequency of testing needs study. Additives need to be evaluated. In particular, additives for grouting may be included to provide a higher assurance of completely filling cells. The guide specifications are a living document that should reflect design and construction aspects for the highest quality, most economical product that provides the intended purpose.

Masonry Details

There are many masonry details in various publications. Unfortunately, there seem to be very few good masonry details. Many have been carried forward without regard to reflecting current design and construction practices. Consequently, the Corps is currently in the process of developing masonry details. These will be available on Intergraph and will be open-ended. That is, details will be added as needed to provide the best detailing information possible to the designer.

Masonry Veneer/Steel Studs

The Corps has placed a moratorium on the use of masonry veneer/steel stud wall systems pending an evaluation of current practices relative to detailing, anchorage, and lateral support. There are specific details required to assure that water is directed to the exterior and the steel stud system is fully protected. These involve brick ledges stepped down from the stud wall level, proper sheathing, water barriers, and flashing. Details to assure adequate anchorage need to be standardized. Lateral bracing systems need to be developed that do not rely on arbitrary connections and member sizes. High walls and multistory buildings need structural steel braced frames or shear walls of masonry or concrete. Adequate guidance is being developed and will be disseminated as soon as possible, probably by the end of this calendar year. Masonry veneer/steel stud wall systems can be used if appropriate design and detailing practices are made available, along with good construction practices.

Research

The Corps has embarked on several masonry research projects through the Construction Productivity Advancement Research (CPAR) program. These are cost shared ventures with private industries and universities. One project involves the development of a lightweight concrete masonry unit that weighs about one-half the current normal weight unit. This is expected to lead to considerable savings through increased productivity. Another project involves testing for basic properties that will be used for establishing development and splice lengths for reinforcing bars in grouted masonry.

Masonry Task Group

A Masonry Task Group was formed to evaluate Corps masonry needs and to provide support and guidance in the development of those needs. Three projects were initiated concurrently. The top priority was to develop masonry design programs. This is an ongoing effort. Guidance for quality assurance and nondestructive testing is complete and has been incorporated into the revised Masonry Manual. In addition to the computer programs, the task group will be supporting the research activities and reviewing and providing guidance on all other masonry activities.

Several members are on national masonry committees and are providing Corps input to codes and standards. This group provides a good forum for continuing the development of guidance to further enhance quality masonry products.

Masonry Course

The masonry course is in its fifth year. It is a 1-week course, taught twice a year with 30 to 35 students, except for 1 year when it was taught three times to satisfy a demand of over 100 applicants. Approximately 350 Corps and non-Corps personnel have attended the course. The course consists of 4 days of classroom work and a one-half day field trip. The field trips are to university laboratories doing masonry research, concrete block plants, and construction sites. Two activities are incorporated into each field trip. The course provides an excellent opportunity for technology transfer, along with design and construction information. The course is a very valuable part of the process for providing quality masonry products to our customers.

Quality Assurance

To assure high quality masonry products, engineering support is required on a continual basis from onset of the design through the completion of construction. The contract documents must be complete and show sufficient details to adequately communicate to the contractor and field quality assurance staff the intent of the designer. Shop drawings must be approved by a structural engineer. To supplement the contract documents, the design engineer should provide instructions to the field to identify those items that are most critical to constructing quality masonry. The final step in the design/construction process to achieve quality masonry construction is site visits by the designer.

Contract plans

The contract plans must show sufficient details to adequately communicate to the contractor and field quality assurance staff the intent of the designer. The extent of detailing needed is different for every building; however, there are minimum contract drawing details required for all masonry construction as follows:

- Typical details for piers, columns, pilasters, and their locations. It must be clear how the typical details are to be applied to all required locations for these elements.
- Control joint details and their locations in the structure dimensioned. Both plan and elevation views are normally needed to clearly show all crack control joints and brick expansion joints.
- Details of horizontal and sloping tops of walls. Include control joints, beam pockets, and method of anchorage for the roof system as applicable.
- Typical details of reinforcement around openings. It must be clear how the typical details are to be applied at all openings.
- Details showing continuity of structural bond beams. Particular attention must be given to achieving continuity in stepped structural bond beams at the tops of sloping walls.
- Details showing intermediate bond beams and how continuity is provided when it is interrupted by openings and corners.
- Details of mechanical openings that may have a significant structural impact.

Instructions to the field

Instructions to the field provide additional assurance for quality products. Masonry construction includes a wide variety of materials, including brick, concrete masonry units, mortar, grout, flashing, reinforcing steel, joint reinforcement, control joints keys, brick expansion joint materials, anchor bolts, etc., all assembled by a mason to form walls with and without openings, columns, piers, and pilasters. Although all of these items are contained in the

contract documents and thus all are important, the quality assurance program does not allow for continuous masonry inspection by Corps personnel. It is therefore imperative that the design engineer provide instructions to the field to identify those items that are most critical to constructing quality masonry. This will allow coverage of the most critical features of masonry construction to take maximum advantage of the time available to the quality assurance personnel. The following items, which represent areas that have caused significant problems on a repetitive basis, are not all inclusive but should be included as critical items in all "Instructions to the Field" lists:

- **Mortar proportions** must be in accordance with the contract. Strength, resistance to water permeance, protection of reinforcement, and durability are derived by the proper mixture.

- **Grout slump** must be in the range specified, and grout must be mechanically vibrated to assure complete filling of cells. Mortar or concrete must not be used in lieu of grout.

- **Reinforcing steel** must be properly positioned and held in place with centering devices that allow for grouting and mechanical vibration. Lap lengths must be as required by the contract drawings. Unapproved interruptions of reinforcing steel for openings must not be allowed. The structural engineer should be contacted when conflicts arise.

- **Air spaces** in cavity walls must be kept free of excessive mortar droppings to allow water passing through the outer masonry wythe to reach the flashing and exit through the weepholes.

- **Brick expansion joints** must be kept free of all material, including mortar, and then sealed with backer rods and sealant. A compressible material installed in the expansion joint for the purpose of keeping mortar out of the joint should not be used.

- **Masonry bonding** at corners is required. This is to provide adequate support to wall elements.

- **Joint reinforcement** must be of the right type and properly placed. One longitudinal wire should be in each mortar bed, normally two in CMU and one in brick. Truss type joint reinforcement should not be used. Factory fabricated intersections and corners are required.

- **Insulation** panels in cavity walls must be in close contact at all edges and tightly adhered to the backup wythe in cavity wall construction to assure the assumed overall coefficient of heat transmission (U-value).

- **Flashing** must be installed so that cells to be grouted are not blocked. It should extend into the backup wythe through the first mortar bed only, lapped and sealed as required. Lapped and sealed joints are especially critical for steel stud backup wall systems. Partial panel length flashing for lintels, etc., should be turned up at the ends.

- **Ceramic glazed and prefaced masonry units** should be set level and true so that the glazed and prefaced facing will present true planes and surfaces free of offsets or other distortions.

- **Masonry unit protection.** Tops of masonry, while being stored and in partially constructed walls, exposed to rain or snow must be covered with nonstaining waterproof covering or membrane when work is not in progress. Covering shall extend a minimum of 2 ft down on each side and be held securely in place. The covering should allow masonry to reach ambient moisture equilibrium.

- **Other items critical to the specific project** should also be included, such as, prism testing of high-strength masonry, etc.

Site visits

Site visits by the designer is the final step in the design/construction process to achieve quality masonry construction. The purposes of these visits by the designer are:

(1) To see that the design intent is being reflected in the construction; (2) To facilitate discussion between construction field office personnel and the designer on special features of the design and critical construction items; (3) To provide feedback on problem areas and design improvements to engineering; and (4) To open the lines of communication between the designer and field so that problems and concerns will be more freely discussed. Although it is recognized that the degree of engineering support during construction is under continual time and cost constraints, the need and value of site visits by the designer has been clearly established in published guidance. Every effort to implement a program of site visits during critical phases of masonry construction should be made, not just in response to isolated field problems. Note that the "Instructions to the Field" will provide an excellent short checklist for the quality assurance staff and for the designer during routine field visits to masonry construction.

Summary

The Corps is currently involved in a very aggressive program in masonry to assure quality masonry products. This program involves revising the Masonry Manual (Departments of the Army, Navy, and Air Force 1982 (Feb, Aug)), updating the masonry guide specifications, developing masonry details, participating in research activities, providing guidance and support through the Masonry Task Group, teaching a masonry course, and participating on national masonry committees. This program, along with developing quality contract documents, with structural engineer approval of shop drawings, providing instructions to the field, and making site visits, is effective in providing our customers with quality masonry products at reasonable costs. The Corps must maintain a leadership role in the masonry industry.

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Collapse of a Long-Span Tensioned Fabric Structure

by
Thomas D. Wright, PE[1]

Abstract

On March 6, 1989, two identical long-span tensioned fabric structures at Fort Leonard Wood, Missouri, collapsed during an ice and snow storm. The structures were constructed using the design-build, request-for-proposal process. The structures were 160-ft square with Tedlar-clad, vinyl-coated polyester fabric tensioned over truss arches. The supporting arches were aligned along the diagonals of the building, meeting at the apex of the three-hinged arch. The arches were supported in a unique way, suspended from wire rope slings attached to tripods at each corner of the building.

The two structures dramatically collapsed within minutes of each other. The failure occurred at loads far below the design requirements. An analysis showed that the failure was caused by buckling of the lower chord of the truss arches. Although tensioned fabric structures are complex, the cause of this collapse was a fundamental structural engineering oversight. Inadequate arch capacity resulted from incorrect computer modeling and failure to analyze unbalanced snow loads. Major design deficiencies, not related to the collapse, which greatly effect the longevity of the building have also been identified.

Introduction

On March 6, 1989, two long-span tensioned fabric structures collapsed during an ice and snow storm. The structures were officially called "Mutli-Use Buildings" an were used primarily for physical training of troops. A Board of Investigation was convened by the District Engineer, US Army Engineer, District, Kansas City, immediately after the collapse. The board included six engineers, and one architect representing Missouri River Division (MRD), Kansas City District (MRK), Construction Engineering Research Laboratory (CERL), and Headquarters (HQUSACE). The primary mission of the board was to determine the cause of collapse of the buildings (Staab 1989). The Board sought and hired an A-E firm experienced in tensioned fabric structures and forensic engineering. The engineering firm of Horst Berger Partners (HBP) was hired by the Board to perform a structural analysis and to provide an independent assessment of the cause of collapse. The primary cause of collapse was the result of a simple structural engineering oversight, but numerous design deficiencies also were found by HBP.

Background

The funding authorization bill for this project included a mandate that required construction of an architectural fabric structure. This project was apparently looked upon as an

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experiment to find inexpensive methods for building large, unheated structures. The request-for-proposal (RFP) design-build process was used to take advantage of industry expertise and economies. A contract was awarded to Brunson Associates, St. Louis, MO, as the prime contractor. Brunson's proposal included construction of two Spandome tension fabric buildings, designed and built by Spandome Corporation of Mountain Lakes, New Jersey. Construction of the two multiuse buildings was finished in 1988.

The Spandome structures were 160 ft square with Tedlar-clad, vinyl-coated polyester tensioned over truss arches (Photo 1). The supporting arches are aligned along the building diagonals, meeting at the apex of the three-hinged arches, forming a crossed-arches arrangement. The arches themselves were formed from box-shaped truss sections fastened together as a series of arc chords to form the arch. The arches were supported in a unique way, suspended from wire rope slings attached to tripods at each corner of the building. The tripods were also trusses formed from steel shapes. The crossed arches supported on the tripods support the tensioned fabric to form an anticlastic dome shape. The side walls are supported at the base of the dome by cables strung between the tripods and propped up by guyed steel posts. Fabric side walls extend to the perimeter foundation and floor from the base of dome (Figures 1 and 2).

Structural System

As described above, the bases of the crossed arches are suspended on wire rope cables from steel tripods located at each corner of the building. Thrust for support of the arch is provided by a wire rope cable which runs around the perimeter of the structure. The perimeter cable passes through the cable saddle provided for the tripod suspension system at the base of the arches. Stability of the entire structure is dependent on the integrity of the perimeter cable. The fabric anticlastic shape (concave in one direction and convex in the orthogonal direction) is achieved by prestressing the fabric in two directions over the arches. The anticlastic shape allows gravity loads to be carried horizontally to the arches and wind loads (uplift) to be carried vertica" to the

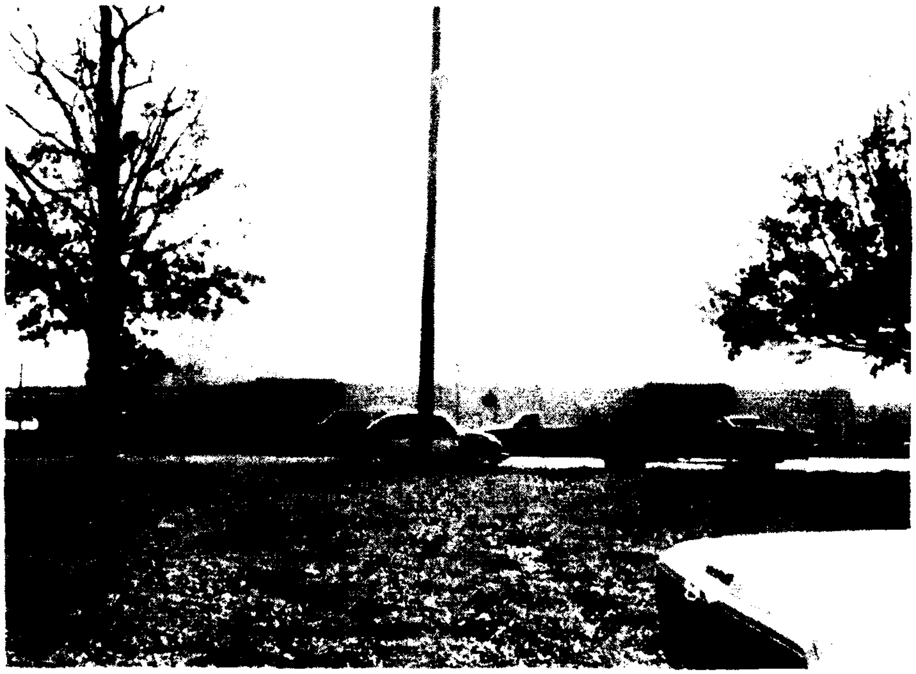

Photo 1. Spandome prior to collapse

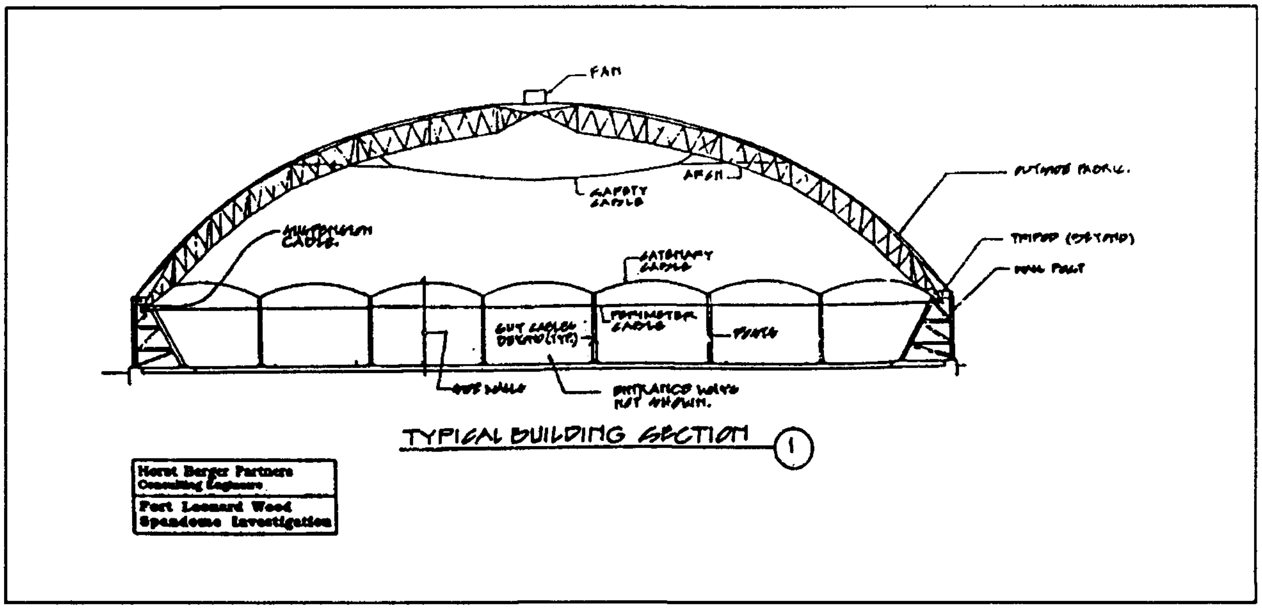

Figure 1. Typical building section

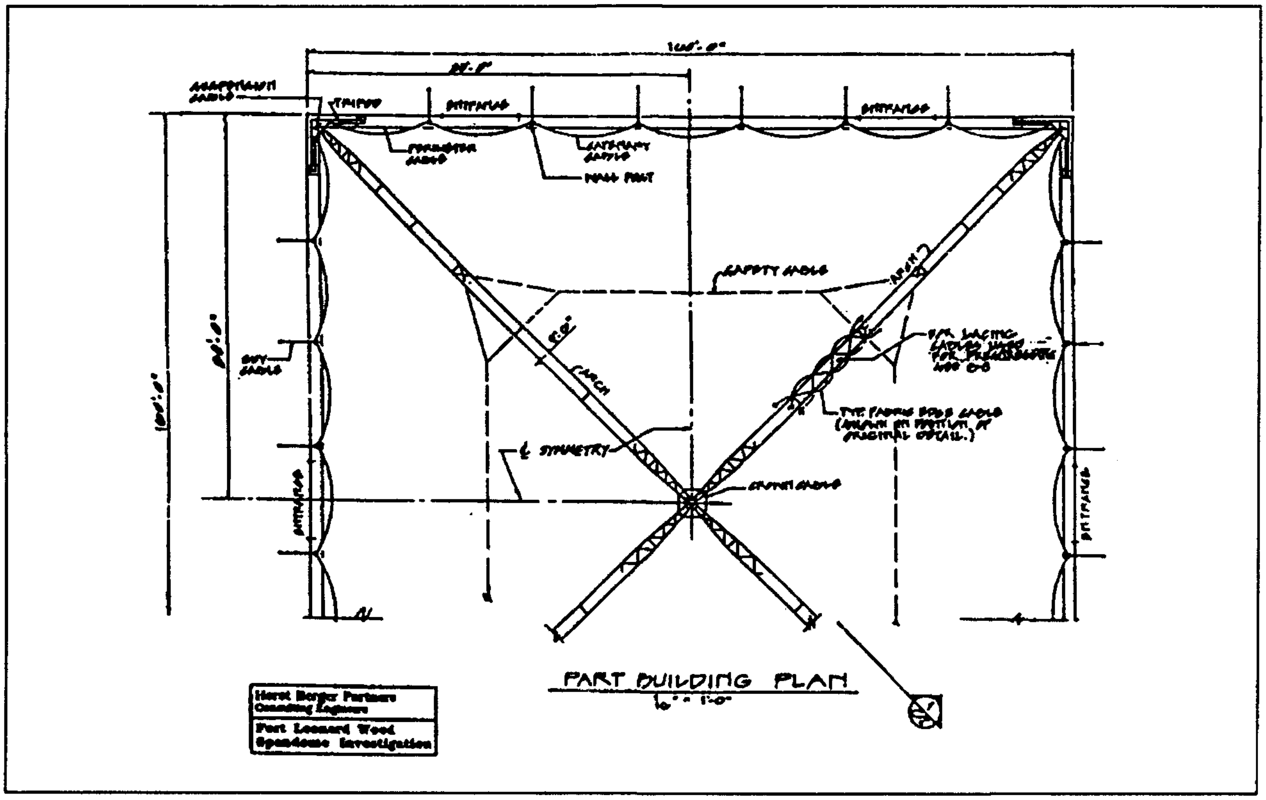

Figure 2. Partial building plan

arches and the perimeter foundation through the side wall guys (HBP (pp 11 and 12)). The fabric behavior is nonlinear and proper design requires accurate determination of physical properties, such as strength and "stretch factors." The properties are usually determined by the architectural fabric fabricator using appropriate testing (HBP p 25).

Description of Collapse

On the morning of March 6, 1989, the two multiuse buildings dramatically collapsed within minutes of each other. A late winter storm began on March 5, 1989, bringing snow, ice, and rain to Fort Leonard Wood. From the beginning of the storm until the structures collapsed, a total of 3.26 in. of precipitation fell in the form of rain, snow, and freezing rain. Snow had accumulated on the structure prior to the collapse, but as discussed later, the load on the structure was far less than the design snow load.

The orientation of the two collapsed structures was amazingly similar, in fact, almost identical (Photo 2 and Figure 3). Three of the four arches buckled on each of the structures. In each case, the southeast leg of the arch was the only leg that did not buckle. In both structures, the most severely buckled arch was located on the northwest corner of the building (HBP p 19). The fabric condition and position were similar on both structures and snow was piled up at the collapsed apex and along the south and east sides of the building (HBP p 7).

Load Analysis

The RFP specification required the structures to be designed for ANSI A58.1 (ANSI 1982) snow loads using a ground snow load of 15 psf. Using A58.1 and accounting for roof slope effects gives a basic design roof snow load of 12.6 psf. ANSI A58.1 also requires unbalanced snow loads to be considered. For the geometry of the Spandomes at Fort Leonard Wood, the unbalanced loads are 6.3 psf at the apex of the arch, 25.2 psf at about the 1/8 of span location, and decreasing to 9.45 psf to the outside edge (eave) of the roof (Figure 4). Spandome's design analysis indicated the structures were designed for a load uniform load of 20 psf (HBP pp 13 and D-5). Spandome also made an unsubstantiated claim that the load at the time of collapse was 30

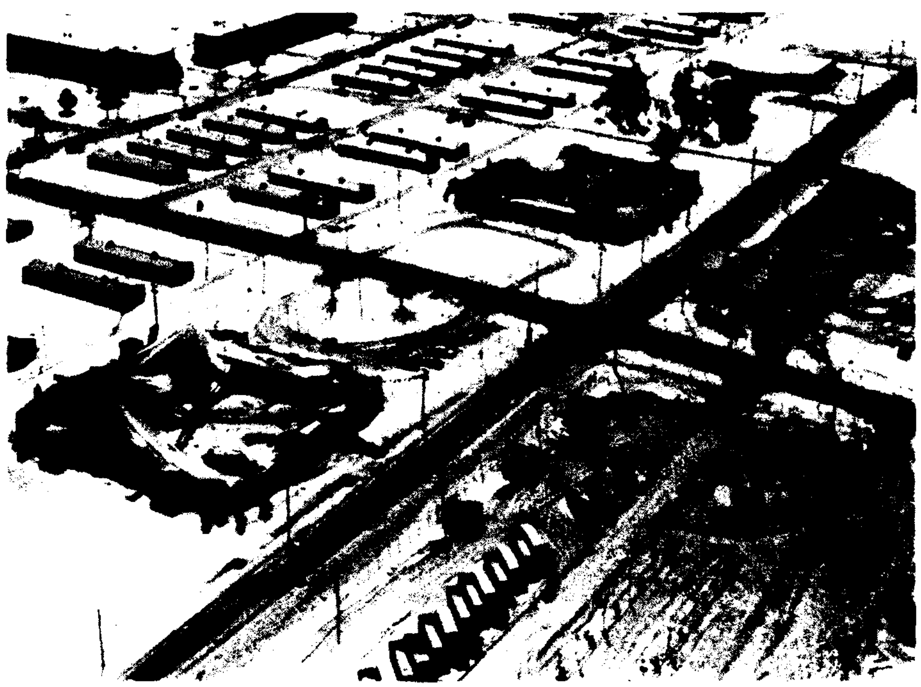

Photo 2. Collapse Spandome buildings

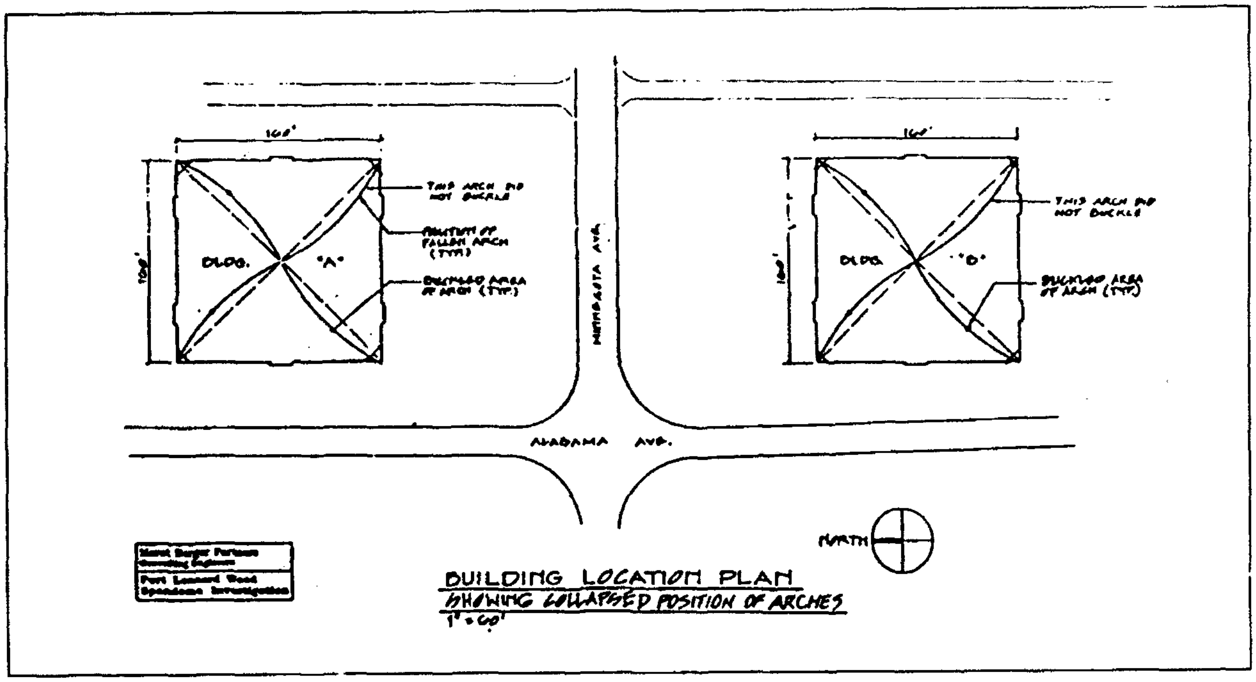

Figure 3. Collapsed position of arches

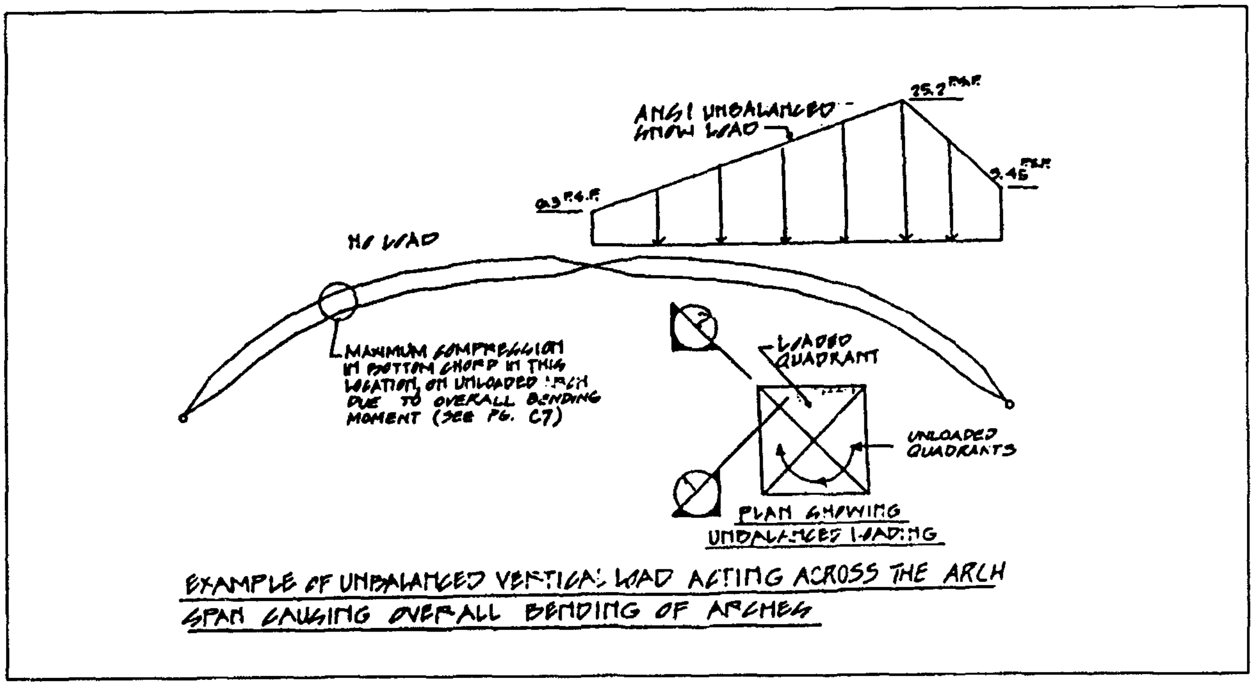

Figure 4. ANSI unbalanced snow load

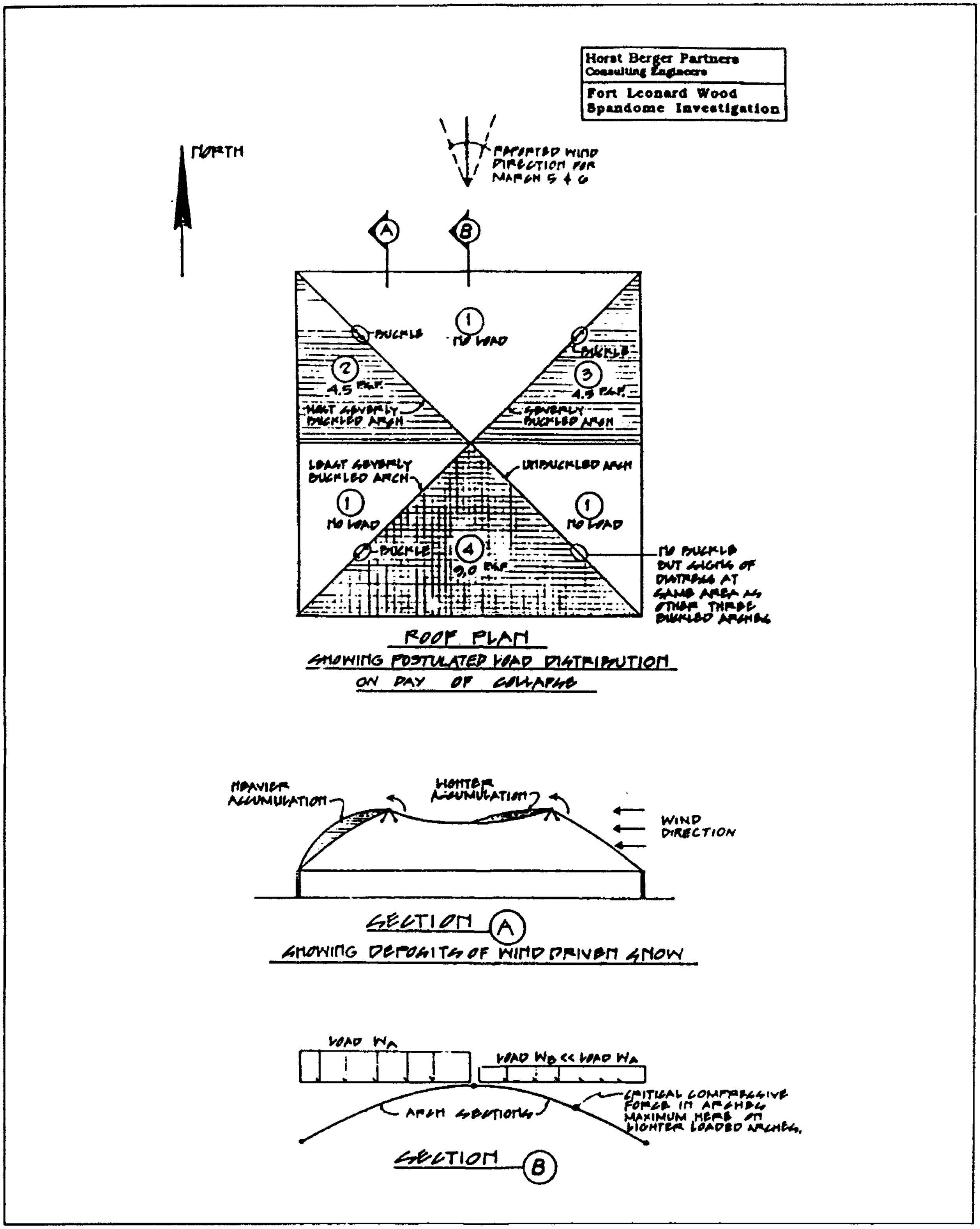

Figure 5. Load distribution on day of collapse

psf (HBP p 9). The Spandome designer had also stated in design documents that unbalanced loads did not have to be considered on a Spandome structure, since they claimed the structure could tilt and maintain symmetrical arch loads (HBP p 14).

An analysis of the weather conditions before, after, and at the time of the collapse showed that the ground snow load was approximately 10 psf. The average roof snow load was calculated to be 7.1 psf, and the winds were from the north to northwest at a maximum of 14 mph. HBP reviewed photographs, slides, video tapes, precipitation data, and wind data to formulate the probable roof load and its distribution at the time of the collapse. HBP concluded that the light winds during the storm caused unbalanced snow loads. The snow accumulated lightly (4.5 psf) on the northern half of the structures and more heavily (9.0 psf) on the southern half of the structures (Figure 5)(HBP pp 17 and C-12). This unbalanced load induces bending moments into the arches and critical compressive forces in the arch legs under the lighter loads, opposite the heavily loaded arches.

Analysis Procedure

A nonlinear finite element model was developed including the fabric, arches, and all cables. The model was loaded with wind and snow loads using fabric prestress levels of 15 and 25 lb per lin in. in two directions. ANSI design balanced and unbalanced snow loads were investigated as well as the estimated load at the time of collapse. The deformed shape of the model under ANSI unbalanced snow loads is shown in Figure 6. Arch loading from the nonlinear model was then applied to a separate, linear model of the arches only. In the arch model, all arch chord and web members were modeled in their as-built locations to determine the effects of local (secondary) bending moments (see Figure 7). The critically loaded chord splices were modeled with both fixed- and pinned-end conditions to bracket the actual stress range. Arch stresses are not sensitive to fabric prestress levels.

Results of Analysis

The investigation revealed deficiencies in the Spandome design analysis. The truss arch members were not truly "trussed out." That is, the members were not configured to carry axial loads only as in a pure truss. Local eccentricities caused by nonconcentric member lines of action were not considered in the Spandome design. Secondary bending moments proved to be critical to the arch capacity. Arch bending moments resulting from unbalanced loads were not considered by Spandome. Forces on the critical truss elements come from three sources: (1) Axial compression results in the arch top and bottom chords due to thrust on the arches, (2) Arch moments due to unbalanced wind or snow loads result in tension forces in the top chord and compression in the bottom chord, and (3) Bending moments are also induced in the splice plates and at the panel points due to the nonconcentric chord forces at the splice (HBP pp 11 and C-7). Failure of the structure was caused by unbalanced loads.

The critical bolted splice in the lower chord of the arch was modeled as both "pinned" and "fixed" at the splice plate. The chord member was considered continuous through the panel point connection (Figure 7).

Stress ratios were computed using American Institute of Steel Construction (AISC) interaction formulas 1.6.1a and 1.6.1b (AISC 1978) and compared to the required maximum stress ratio of 1.0. Considering axial loads only (ignoring effects of local bending moments) computed stress ratios are 0.76 for the chord at the splice (or kink point), 0.95 for the splice plates, and 0.80 for the chord at the panel point applying the uniform load case. The structure satisfies the AISC interaction equations if only the uniform load case is considered and ignoring the effects of local bending moments. For the ANSI A58.1 unbalanced snow load case, the axial stress ratios range from 1.27 to 1.59. Stress ratios for axial forces only using the formulated load at the time of collapse range from 0.72 to 0.90 (Figure 8).

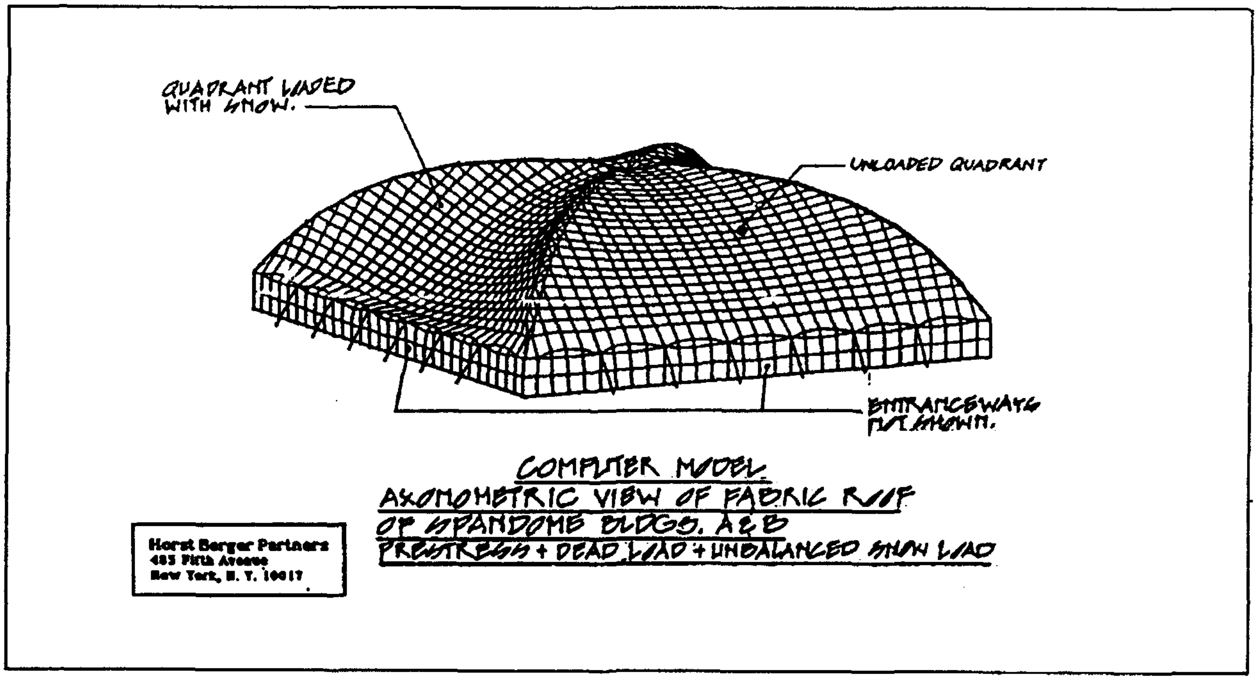

Figure 6. Computer model of fabric roof

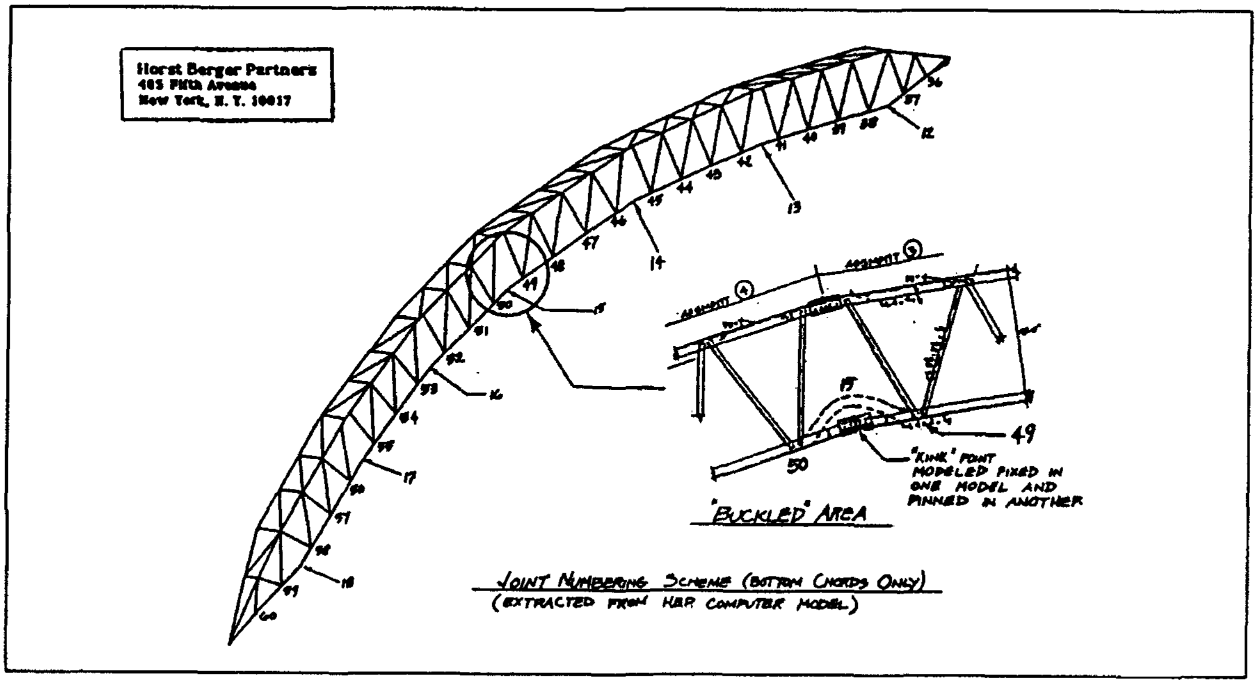

Figure 7. Computer arch model

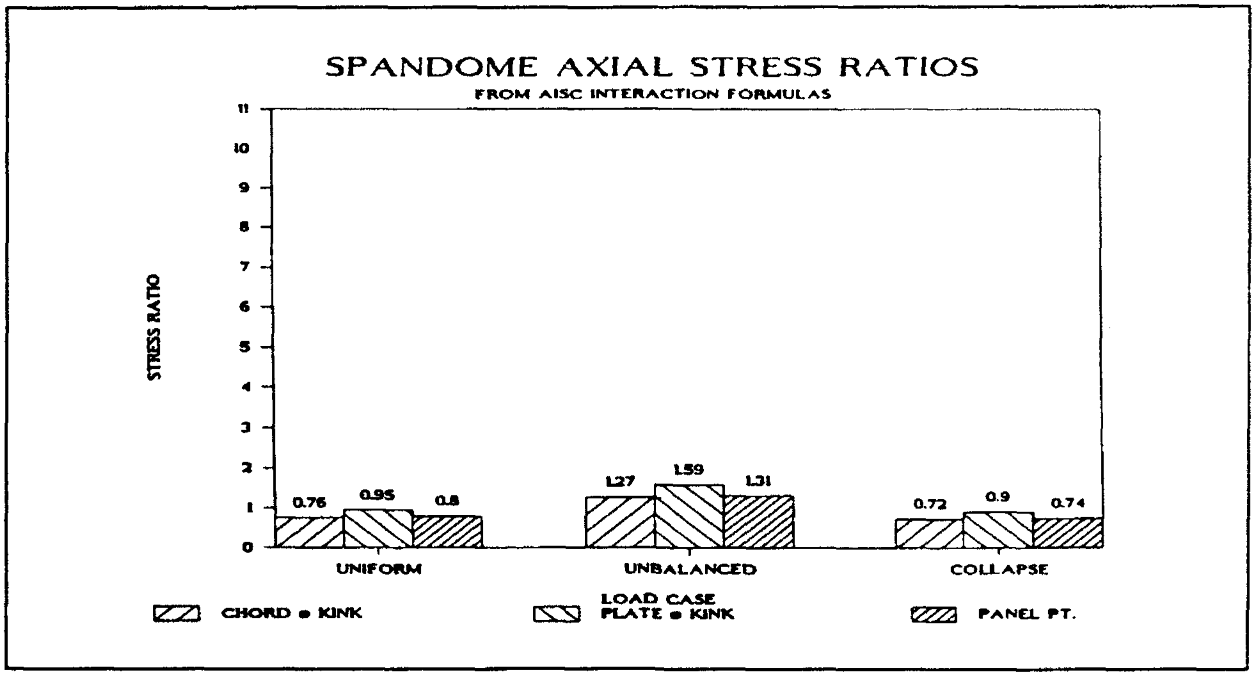

Figure 8. Spandome axial stress ratios

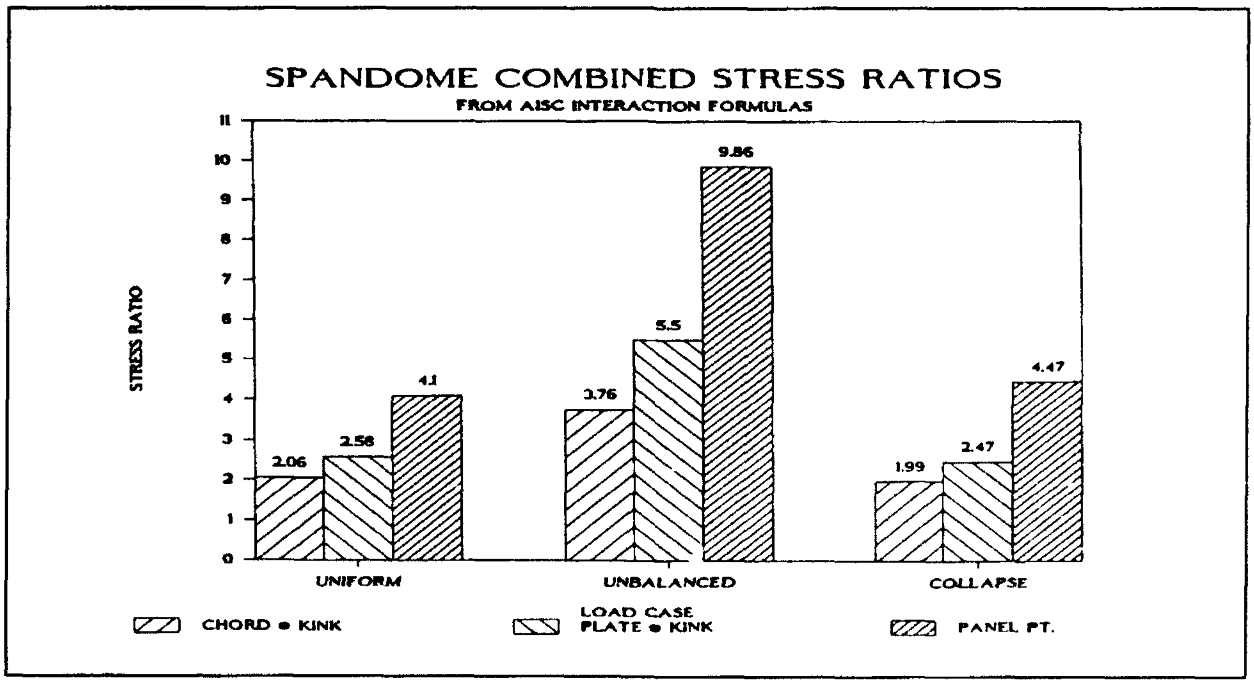

Figure 9. Spandome combined stress ration

Including the effects of local eccentricities changes the stress ratios dramatically. Under the design uniform load, stress ratios range from 2.06 to 4.10. The ANSI unbalanced load results in stress ratios ranging from 3.76 to 9.86 for the different locations. The stress ratios were computed to range from 1.99 in the chord at the splice to 4.47 at the panel point using the load at the time of collapse (Figure 9)(HBP pp E-8 to E-26).

These extremely high stress ratios resulting from local bending moments obviously predict failure.

Other Problems

Horst Berger Partners identified a number of design problems with the structures, yet unrelated to cause of the collapse (HBP pp 21 to 28). A partial list includes:

- The arch diagonals in the vertical plane were overstressed by 30 percent for the unbalanced snow load case.
- Chord members were overstressed in areas other than the failure point.
- The safety cables were ineffective in stabilizing the arches unless the fabric failed.
- The Spandome analysis was too simplistic for this complex nonlinear structure.
- The Spandome arch model did not consider the effects of local bending.
- Spandome's statements that unbalanced loads need not be considered are unsubstantiated.
- Arch and fabric deflections were not considered.
- Fabric stresses were calculated only at the arches.
- An undetermined amount of fabric prestress was used.
- Poor fabric connections were used resulting in fabric tearing and cable damage.
- Connection details used resulted in cable and fabric abrasions.
- American Iron and Steel Institute required cable saddle radii were not used, shortening the cable life.

Conclusions

The Board of Investigation concluded that the structure collapse was caused by inadequate arch capacity resulting from incorrect computer modeling and failure to analyze unbalanced snow loading. Incorrect computer modeling resulted in a failure to account for local bending moments present in the as-built structure. Failure to analyze unbalanced snow loads resulted in omission of arch bending moments from the analysis. Critical detailing deficiencies existed on the Spandome structure which would have severely shortened the structure life. Spandomes lack redundancy and are vulnerable to sudden catastrophic collapse. Critical deficiencies make Spandomes an unacceptable structure (Staab 1989).

Lessons Learned

Design-build contracts must be subjected to the same rigorous design review as A-E designs or in-house designs. When an A-E is hired to design a project, the A-E warrants that design to the government. When the government contracts for construction of that project, the government implies a warranty for the design to the contractor. The A-E may ultimately be held responsible for any costs incurred by that warranty, but the government incurs a potential liability in the construction contract. In the design-build contract relationship, the government does not have the same liability for the design. None the less, the Corps has a responsibility to our customers, as his agent and as professionals. Unusual or high risk structures, such as very long spans or structures without redundancy, should get special review by qualified experts. This could be done by a highly qualified A-E or by a peer review panel or board.

The importance of detailing and assuring that details match the design assumptions cannot be overstressed. Improper design assumptions and detailing resulted in the collapse of these structures.

Unusual structures require specialized expertise for design and review. These structures had major problems, outside of those that caused the collapse, that would have greatly reduced the serviceability and longevity of the buildings. Review by designers experienced in fabric and cable structures could have reduced or eliminated these problems.

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Wind Damage at Fort Hood

Joseph P. Hartman, PE[1]

Abstract

During May 1989, Fort Hood, Texas, experienced a storm with winds exceeding the design wind speed. This storm caused significant damage across a wide area of the base and surrounding communities. Most damage was limited to roofing materials, but there were several instances of damage to roof structures, roll-up doors, metal walls, and masonry walls. Inspection of the damage led to several lessons learned: 1) Provide better attachments to hold down gutters and eave flashing; 2) Avoid use of tall unreinforced masonry parapets; 3) Be cautious in using unreinforced masonry walls; 4) Design walls under joist seats for upward loads due to roof suction; 5) Use vertical lift doors if funds are available, since roll-up doors have a high damage potential; 6) Emphasize construction quality control, local deficiencies can provide weak points which can initiate large failures.

Introduction

On 13 May 1989 a storm hit Fort Hood, Texas, with winds near the design velocity. Various facilities were damaged mainly by loss of roofing materials, but there was also some more serious damage. US Army Engineer District, Fort Worth, and the area office immediately initiated a clean-up and repair effort. On 24 May 1989 US Army Engineer Division, Southwestern, also sent a team to inspect the damage. The purpose of this inspection was to identify design or construction practices which may have contributed to the damage and to recommend changes to reduce future damage potential. The inspection concentrated on newer, permanent facilities, plus minor attention to family housing.

Wind Speed and Design Pressure

On 13 May 1989 winds at Fort Hood Airfield reached a sustained speed of 60 knots (about 70 mph) out of the northwest, with gusts up to 85 knots (100 mph). Gusts at Gray Army Airfield (approximately 10 miles from Fort Hood Airfield) reached only 61 knots (70 mph). The difference between the two airfields indicates the localized nature of peak winds. Actual wind velocities at each damaged facility can not be determined, though much of the damage occurred nearer to Fort Hood Airfield.

Current criteria for wind design in TM 5-809-1 give 70 mph as the basic wind speed for Fort Hood. This represents a 50-year event of the "fastest-mile" wind. The square of the basic wind speed is multiplied by a gust response factor of about 1.25, depending on height, representing an equivalent gust speed of about 78 mph. Each building is assumed to have an open exposure upwind (exposure C). These criteria are essentially the same as ANSI A58.1-1982 and ASCE 7-88.

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The wind speed is converted into design pressures by the formulas in the TM. These produce different pressures for main framing elements and cladding elements, as well as identify localized high-pressure locations such as eaves and corners.

Description of Damage

Most facilities at Fort Hood received no damage. Those buildings that were damaged experienced, for the most part, loss of roofing materials. Some facilities sustained such damage as collapsed masonry, partial loss of roof structure, or inoperable roll-up doors. A description of each inspected building is included in Appendix A. The level of damage to these buildings was similar to that experienced by commercial and residential buildings in the surrounding communities.

Many buildings experienced partial loss of roofing materials. Damage seemed to have consistently started at the windward eave. Air flowing up the side of the building lifted the eave flashing or the gutter. Once these lifted, they started peeling back the adjacent roofing and insulation. This occurred for roofs with adhered rigid insulation, mechanically fastened insulation, and screwed-down, standing-seam roofing. In the latter two cases, the screws usually pulled out of the metal deck into which they had been fastened. If the gutter or eave flashing had been fastened better, much of this damage may not have occurred.

Several large barracks built around 1973 had structural damage to the roof, in addition to the roofing damage described. Suction on the windward edge of the roof lifted a portion of the deck and joists and bent it back. Generally, the welds between the metal decking and the bar joists were strong enough to lift the joists off the bearing wall, though some deck welds were broken. Failures at the joist bearings occurred sometimes through the attaching welds, the welds between the bearing plate and anchors, or due to anchorage pull-out or lifting of the bond beam off the supporting wall. There was no consistent mode of failure, thus no obvious weak link. Though most inspected connections looked adequate per criteria, several joist-to-bearing plate welds were of poor quality. At the corner of one damaged area there was no evidence of any weld on the last joist seat. This may have allowed the joist to lift and initiate a progressive failure of the roof structure.

There were very limited areas where the metal deck had pulled loose from the joists. Welds between decks and supporting joists were mainly of good quality. However, as might be expected, certain individual welds were inadequate. On one short joist there was no evidence of any deck welds. Despite overall good quality, isolated weak spots may permit initiation of progressive failure.

Damage to masonry occurred on several buildings. Some windward parapets were blown over; none had vertical reinforcement. Calculations for one 12-in.-thick parapet showed that it should have withstood the wind pressures if the bedding joints had any mortar bond strength but would have toppled without some bond strength. This indicates that mortar bond probably did not exist, either due to flashing materials, or to joint deterioration due to weather or thermal movements. In other buildings, bond beams were damaged or pulled off where roof joists were lifted. On one older building, entire sections of unreinforced masonry wall pulled away from the steel frame and then collapsed. Several clips between steel columns and undamaged walls of this building were loose, possibly indicating the cause of the wall failures. On another building, one masonry wall extending a few courses above a lower roof tipped over due to lateral forces from an attached upper wall panel (see Figure 1). The vertical reinforcement in this wall extended up to the lower roof only, even though the drawings had shown it extending to the top of the masonry. More effective use of vertical reinforcement would have prevented some of the masonry damage to the buildings.

Many roll-up doors were damaged by the wind, but these were very small in number compared to the undamaged roll-up doors.

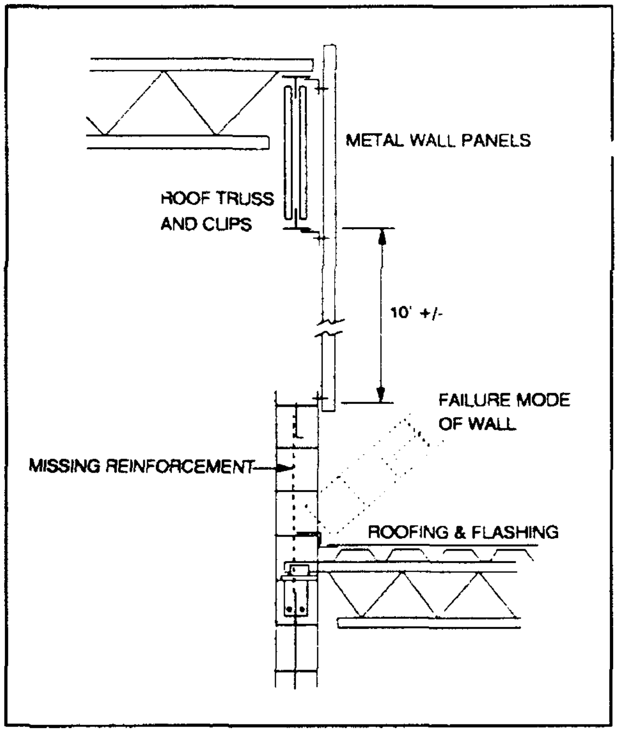

Figure 1. Wall section for Building 3850

Damage occurred to doors from 14 ft to 28 ft wide. Damage was, in all probability, initiated by catenary tension pulling the retaining clips out of the door tracks. The door panels were presumably bent after the clip failures. There were no observed failures of door track attachments to the building frames or CMU walls.

Recommendations

Sheet metal fascia and gutter detailing seemed to be a weak link that led to further damage to roofing and roof structure. CEGS 07600 has optional statements that would require gutter brackets at 3-ft spacing. These requirements should be included for all buildings, and are especially important for tall buildings. These brackets should be screwed to solid timber nailers and should also attach to the outer edge of the gutter to hold it down. Where a fascia strip is used without a gutter, the lower edge of the sheet metal should be held in place with a continuous lock-type seam or with sufficient screws through slotted holes which allow minor thermal movement.

Most roofing damage apparently started at damaged fascia and gutters, since roofing damage was not apparent where parapets were used. Once damage is initiated, no type of attachment is likely to prevent additional loss of roofing. Current roofing practice is probably adequate. However, to obtain an adequate uplift rating on standing-seam roofing, the clips or subpurlins must be screwed into 16-gage minimum thickness members, not into a light-gage metal deck. (More complete guidance on this is being provided in a draft ETL on standing-seam metal roof systems.)

The several types of masonry failures which occurred could probably all have been prevented by vertical reinforcing. However, this adds to construction costs, and contractors in some regions are not experienced in extensive use of vertical reinforcement. Designers should consider adding reinforcement at least in CMU parapet walls and as tie-downs for bond beams on high eaves.

Roll-up doors of all sizes experienced varying degrees of damage. Wide roll-up doors also require frequent maintenance under normal conditions to keep them operating smoothly. The worst damage occured to 28-ft-wide doors, while the same size vertical lift, sectional doors in an adjacent building were not damaged. Use of vertical lift doors would prevent damage, but at a considerable increase in cost. CEGS 08330, March 1989, calls for roll-up doors to be tested and certified for 16 psf, plus a 1.5 safety factor. These pressures should be adequate, at least in moderate wind areas, *if the supplier fully complies with the* requirements. An arbitrary increase in design pressure would result in increased construction costs.

Appendix A - Observations

7021 AIRCRAFT MAINTENANCE HANGAR

This hangar had no structural damage. Flashing on the windward edge of the roof was bent up or pulled off. Some built-up roofing was pulled off the insulation. Some insulation was pulled off the metal deck, mainly because the fastening screws pulled out of the metal deck. Screws were spaced at about 3 ft in both directions. Several nearby buildings had damage to windward gutters and fascia.

7025 STORAGE BUILDING

This is an older steel frame building with CMU curtain walls. The nominal wind direction was toward the NW corner of this building, which has one long wall facing WSW. However, large hangars a short distance upwind may have complicated wind flows around this building. On the wall facing WSW, a section of 8-in. CMU had collapsed. The section was about *18 ft tall and three* 24-ft bays long with no vertical reinforcement. The wall probably would have vertical reinforcement if designed to current criteria for wind and masonry. Debris had been removed at the time of inspection, but one occupant said the collapse had been outward. Clips between steel columns and remaining CMU walls in the building were of a single-anchor bolt type, and several had rotated downward and loosened. A 16- by 16-ft door on each endwall had also been pulled off and one of the doors reportedly fell inward, possibly causing a sudden increase of internal pressure.

23001 ABRAMS FIELD HOUSE

Windward flashing and gutter had been damaged. Occupants said 600 sq ft of roofing had been lost, exposing the metal deck.

27002 ENLISTED BARRACKS

This is a three-story barracks built about 1973. Walls are CMU with brick veneer, with no vertical CMU reinforcement. The roof is bar joist, with metal deck, adhered 2-in. rigid insulation, 2-in. foamed-in-place insulation, plus sprayed-on vinyl coating. The foam insulation had blistered extremely over most of this roof. Some stone coping was missing on a few parapet walls, the coping had not been doweled to the walls. At the windward edge of the roof, joists bear on a bond beam at the top of the CMU, there is no parapet, the roof drains into a large exterior gutter, and the roof is high above any upwind obstructions.

Windward gutter damage was apparent over much of the building. A large area of roofing at the windward edge had been pulled off, down to the decking. About six bar joists had been pulled up from the exterior wall and bent back over the roof. No decking had been pulled up while joists remained in place. A variety of failure modes was apparent at the exterior joist bearings. On several joists the welds to the bearing plate had broken; these welds appeared to satisfy specified requirements (two welds, 1 by 1/8 in.). One bearing plate had broken loose from the welds to the anchor studs. One bearing had pulled the anchors out of the bond beam. One bearing plate, adjacent to a building expansion joint, showed no evidence of welds to the joist. One joist had one seat angle missing, and it appeared there had been no weld fusion between the angle and the joist web bar. Portions of the bond beam had been pulled up with the joist bearings. Bond beam reinforcement was discontinuous at a control joint. There was some additional masonry wall damage at these bearing locations and at the interior bearing wall.

High wind speeds and bad exposure probably caused unusually high suction over part of the roof. Failure of the roof structure may have started at the missing joist bearing welds and progressed to adjacent joists.

34010 ENLISTED BARRACKS

This building is similar to building 27002. Bad roofing blisters were not apparent here. The pattern of damage was similar to 27002 but

debris had been removed. Very open ground extends miles upwind from the damaged area. Most welds between deck and joists seemed sound. A ridge member solidly welded between abutting joists had been pulled loose when the joists were lifted by the wind. This indicates significant force exerted by the wind.

34006 ENLISTED BARRACKS

This building is similar and adjacent to building 34010. Most damage was limited to the windward gutter and flashing, but this had progressed to pull up the roofing materials. The bottom 2 in. of insulation was rigid glass. Near the north end of the west wall a row of deck welds next to the wall had broken, but the deck was not pulled off. One joist seat under this portion of the deck showed evidence of poor welds. Over the adjacent stairwell, several deck panels had blown off the joists. Most deck welds seemed good, but one joist showed no evidence of any welds to the deck.

3850 TACTICAL EQUIPMENT SHOP

This is a new structure with steel framing (not preengineered), sandwich wall panels with a 14-ft CMU wainscot, and a roof consisting of trusses, bar joists, metal decking, purlins, and standing seam roofing. Vehicle doors are 28-ft-wide roll-up doors. The building has two wings at right angles to each other.

Almost all vehicle doors had been partially pulled out from their frames. One side of most of the doors was pulled outward and one side inward. No door frames were pulled off the structure. Across the street a similar facility (building 4351) showed no damage to its 28-ft-wide sectional overhead doors.

Windward gutters and fascia had been damaged or pulled off. Some roofing was lifted off by failure of the purlin attachments, where the screws had pulled out of the metal deck. Screw spacing was about 5 ft by 18 in. Debris also showed that some clips had pulled out of the roofing seams.

One endwall, at about 45 deg from windward, had failed above an adjacent shed roof, up to the high roof level. This wall consisted of about four courses of CMU cantilevered above the shed roof, a channel anchored to the CMU, and sandwich wall panels attached to the channel and to the truss bottom and top chords. This wall also had two diagonal braces extending from the channel to the bottom of the truss. One brace pulled loose from the channel when the wall failed, and one brace pulled loose from the truss. The vertical reinforcement in the CMU did not extend above the lower roof, there was no bond-beam reinforcement at the top of the CMU, the channel was not structurally attached to any steel column, and there were no columns along the end wall, except at the corners. The wind pressures probably caused failure of the cantilevered CMU, thus initiating the failure. It appears the cantilevered CMU was not adequate for design loads. (Subsequent examination of design drawings showed vertical reinforcement extending to the top of the cantilever CMU wall. It is not known why construction did not conform to these drawings.)

9112 TACTICAL EQUIPMENT SHOP

This is an older steel frame building with CMU walls with no vertical reinforcement. At several locations 12-in. CMU parapets extend four courses above the roof line. Two windward parapets had fully or partially collapsed. The parapets were at right angles to each other, the wind direction was between the normal to each wall. Extensive horizontal cracking of mortar joints was visible for another 6 ft below the fully collapsed parapet. Cracks were also visible at the corners of the building, probably due to past foundation movement. Two 16-ft-wide roll-up doors had been pulled out of their tracks. The retainer bar was missing in both of these tracks; it was not obvious whether this had happened before or during the recent high wind. (Subsequent analysis showed that calculated wind pressures would cause overturning of the parapet if there was no mortar bond strength. Any

mortar bond should have been able to resist failure.)

9122 TACTICAL EQUIPMENT SHOP

This building is similar and adjacent to building 9112. Similar parapet damage occurred. Several 16-ft doors had been partially pulled from their tracks.

40001 TACTICAL EQUIPMENT SHOP

This building is quite old, with cement-asbestos wall panels. The only apparent damage was to windward gutters and roofing.

41010 COMPANY ADMINISTRATION

This low, single-story building experienced damage to windward gutter and fascia, plus some roofing damage.

42000 NCO OPEN MESS

This is an octagon-shaped building located with full up-wind exposure. The windward gutter was ripped up and some adjacent roofing was damaged.

COMANCHE VILLAGE III, FAMILY HOUSING

There was extensive damage to windward shingles on some roofs, minor damage to most others. It appeared that shingles had been applied directly to the plywood deck with no roofing felts. Some fascia and siding had also been torn from portions of some buildings. In several cases building paper was intact where siding had been torn loose. No structural damage was observed.

MONTAGE VILLAGE, FAMILY HOUSING

This is an older area of housing near Gray Army Airfield. No damage was observed.

Structural Problems in the Pacific Ocean Division

by
Arthur T. S. Shak[1]

Abstract

a. **Project**: *FY88 NAF Project, Bowling Center, Camp Foster, Okinawa, Japan.*

Problem: *Lateral restraint of pile foundation.*

Discussion: *In this turnkey project, the solicitation structural requirements stated that the criteria and design of the building for seismic loads shall be in accordance with NAVFAC P-355 (TM 5-809-10) and that the seismic Zone shall be Zone 4. The Contractor proposed to resist the forces in the north-south direction by passive earth pressure acting on the grade beams. This was unacceptable to POD. The Contractor was directed to interconnect the piles with foundation ties in the north-south direction, which he did under protest. The Contractor has since submitted a claim for additional compensation.*

Resolution: *Pending decision by the Contracting Officer.*

b. **Project**: *FY84 MCP, Permanent Capability Facility (PCF), Misawa Air Base, Japan*

Problem: *Extensive cracking of reinforced concrete roof slab.*

Discussion: *Reinforced concrete flat plate slab serving as roof exhibited extensive cracking upon stripping of formwork. Reinforcing steel rebars were misplaced over columns, and deviated substantially from conditions assumed for the design.*

Resolution: *Building Contractor decided to demolish the existing roof slab and columns, and reconstruct new columns and roof slab.*

Opening Remarks

Good afternoon, fellow structural engineers. My topic for this afternoon is "Structural Problems in POD." In my preparation, I encountered some difficulty in finding appropriate material to capture and retain the attention of an esteemed audience like you with the selected topic of "Structural Problems in POD." Therefore, I decided to intersperse this presentation with some quotes from a recent newspaper article authored by one Dr. Robert J. "Call me Bob" Kriegel who describes himself as a "performance psychologist." Mr. Kriegel wrote a book appropriate for these trying times titled, "If it ain't broke...BREAK IT!" I would like to relate some quotes from this book review hoping that you might find them as enlightening as I did.

Having spent a good part of his career fine-tuning the brains of Olympic athletes, Mr.

[1] Structural Engineer, US Army Engineer Division, Pacific Ocean; Fort Shafter, HI.

Kriegel was convinced that the same mental aerobics he was teaching to runners and skiers would work in the business world.

His book captures his experience as a professional motivator and promises to teach us "unconventional rules for breaking out of old modes and mindsets so that we can take effective risks, constantly innovate and continually be on our edge."

What we have to do is break with convention and instead practice "unconventional wisdom."

Kriegel's brand of unconventional wisdom means playing by a different set of rules instead of meeting the competition head on, the way Apple Computer changed the game instead of buying into IBM's definition of personal computing, for instance. Or the way Dick Fosbury, creator of the "Fosbury Flop," changed the rules of high jumping by hopping headfirst over the bar.

Now for my assigned topic. I encountered further difficulty in finalizing this presentation because POD does not have too many structural problems. However, searching through my files, I managed to zero in on two topics to fulfill the "Structural Problems in POD" assignment. Both are from our Japan District, and occurred during project construction.

Foundation Ties

Project: FY88 NAF Project, Bowling Center, Camp Foster, Okinawa, Japan.

Problem: Lateral restraint of pile foundation.

Discussion: In this turnkey project, the solicitation structural requirements stated that the criteria and design of the building for seismic loads shall be in accordance with NAVFAC P-355 (TM 5-809-10) and that the seismic Zone shall be Zone 4. The Contractor proposed to resist the forces in the north-south direction by passive earth pressure acting on the grade beams. He based his proposal on his interpretation of statements in the TM which allows the omission of ties between pile caps in seismic Zone 1 when adequate passive resistance exists. The Contractor's interpretation was unacceptable to POD. The Contractor was directed to interconnect the piles with foundation ties in the north-south direction, which he did under protest.

The Contractor subsequently submitted a claim for additional compensation, stating that the TM allows him an alternate to ties by "overexcavating the soil under the footings and recompacting to control differential settlements and to increase passive resistance so as to eliminate the need for footing ties." Our reading of the TM was that this alternative is permissible only in Zone 1.

This matter was referred to the Contracting Officer for resolution.

Reinforced Concrete Flat Plate

Project: FY84 MCP, Permanent Capability Facility (PCF), Misawa Air Base, Japan.

Problem: Extensive cracking of reinforced concrete roof slab.

The reinforced concrete flat plate slab serving as the roof of this structure exhibited extensive cracking around most columns upon stripping of formwork. Field investigations showed that reinforcing steel rebars were misplaced over columns and deviated substantially from conditions assumed for the design.

Resolution: After lengthy discussions, Building Contractor decided to demolish the existing roof slab and columns and reconstruct new columns and roof slab.

Detailed Discussion: On 13 Aug 1985, my boss received a phone call from the Chief of Engineering Division, Japan District, regarding serious cracks in the roof slab of the subject project. Main points of discussion were that the PCF building roof slab showed many cracks upon form removal. It was reported that the rebars in the slabs over the

columns were not correctly placed; a punching shear problem was suspected. On 20 Aug 85, we received documents relating to the roof slab failure, with the request that we evaluate and comment relating to the adequacy of the AE's design analysis.

Documents from JED included studies by the AE (DMJM), JED's QA Branch engineers, and data from engineers retained by the Contractors. We evaluated submitted documents and concluded that the design analysis prepared by the AE was satisfactory, and the major discrepancy was the rebar misplacement noted from field measurements.

On 27 Aug 85, JED notified us that a decision had been made to remove roof slab and columns all the way to bottom of columns.

In sum, POD's involvement in this matter was to review the AE's design analysis and provide JED with our professional judgment on the adequacy of the AE's calculations. Our judgment was the AE's calcs were satisfactory. To provide this judgment involved review of many documents. This issue became moot when the Contractor decided to rebuild the roof structure, with some modification to the column capitals, at no additional cost to the Government.

Further details as to the origin of this problem and details regarding the evaluation process are included here as Appendix A to this presentation. It includes a detailed discussion of the initiating phone call, evaluations done by the Architect Engineer design firm, the construction quality assurance people, and Japanese engineers retained by the Contractor. The Japanese engineers, by the way, did not concur with the AE and POD that the design meeting the ACI code was adequate.

Closing Remarks

I have presented two examples of structural problems that have occurred in the Pacific Ocean Division. I would like to close with more excerpts from Dr. "Call Me Bob" Kriegel's book: "If it ain't broke...BREAK IT!":

Kriegel urges us to slaughter sacred cows, to have the guts to take risks (not chances) and to conquer the fears that keep us from taking such risks. He believes that "mistakes are a good investment," and that playing it safe is actually dangerous because it practically guarantees we will be left in the dust by less risk-averse competitors.

He believes passion for life and career is the critical personal and professional success factor—not education, not experience, not connections. If you do not have passion for what you are doing, you may as well get out, because the world truly belongs to people with passion and a willingness to follow their dreams.

That's not easy, Kriegel admits, because there's no shortage of people waiting to "firehose" your ideas. You know them. They are the ones who are quick to tell you, "It's not in the budget" or "That's not the way it's done around here." They are quick to douse a hot idea instead of stoking it into a raging fire.

"It ain't broke, don't fix it" thinking breeds a false sense of satisfaction. It causes you to behave like the old story of the well-fed frogs soaking in pots filled with the warm, soothing waters of success. Like these frogs, by the time you realize the environment has changed, the water is boiling and you're too weak to climb out.

I know there is a lesson to be learned somewhere here. Whether it is searching for solutions to structural problems or any other of life's multiple problems, perhaps we should ponder Mr. Kriegel's unconventional wisdom *remarks, and at appropriate times,* plunge head first like the high jumper. It may work at times; at least it did, for the high jumper.

Thank you.

APPENDIX A

TELEPHONE CONVERSATION RECORD 9/14/85

Project Ladylove, Main PCF Building

H Kobayashi/A Shak	Pac Ocean Div, Tech (PODED-T)	438-9552
J Ball/ T Kishimori	Japan Engineer Dist (POJED)	233-3853

1. The reinforced concrete flat plate slab serving as the roof of the subject building was recently completed. Roof dimensions are roughly 200 feet by 200 feet, with slab thickness of 7 inches. Added 8–inch slab thickness measuring 18 inches square (referred to as column capitals) are provided at each column.

2. Upon stripping of formwork, numerous cracks were observed throughout the slab. Before form stripping, there was no evidence of cracking. Initial guess by JED: punching shear problem. AE (DMJM) verified that punching shear was considered in design. Reinforcing steel placement not exactly as design shows. Both AE and JED's QA personnel believe this misplacement of reinforcing a contributing factor to the crack problem.

2. Contractor claims he is used to much bigger capitals than what we show. He has asked for and will be furnished the AE's design analysis by JED.

3. Concrete cylinders tested at 3600psi. The one core sample taken tested out at 2700 psi. This is below the design requirements, but is not considered critical.

5. JED needs a determination whether any design error occurred. They will send us documents including photos relating to this problem for our detailed evaluation .

6. J Ball stressed the critical nature and importance of this project. A Shak responded we will give this problem our immediate and best effort.

* PROJECT LADYLOVE; PCF BUILDING; CRACKS IN REINF CONC FLAT PLATE ROOF SLAB
  * Background:
    * 8/13/85: Phone call from Chief, Engineering Division, Japan District, to Chief, Tech Engineering Branch, POD.
    * Summary of telecon: PCF bldg roof slab show many cracks upon form removal. Rebars not placed correctly; punching shear problem suspected.
    * Packet from Japan Engineer District (JED) received 20 Aug 85. Packet consisted of following:
      * 1. REPORT ON LADYLOVE ROOF CRACKS, 6 Aug 85, AE Daniel, Mann, Johnson, Mendenhall (DMJM) Phillips/Matsumoto
      * 2. ROOF SLAB CRACKING, LADYLOVE, PCF, MISAWA, POJCD-Q, Seto, Quality Assurance Branch, Corps of Engineers, 9 Aug 85
      * 3.. INVESTIGATION REPORT FOR CRACKING AT ROOF SLAB IN MAIN PCF BLDG, LADYLOVE PROJECT, MISAWA AB, 10 Aug 85, by Reiji Tanaka, for the Contractor.
    * Message R 231700Z Aug 85: Kelly AFB to POJED-M Info POD
      * 1. New concr roof of PCF bldg had spider-web type cracks; serious problem.
      * 2. Inspection indicated tension cracks due to negative moment occur at all cols.
      * 3. Test cores show tension steel incorrectly placed; Corps asked contractor for corrective action.
      * 4. Concerned about cracks/delay. Also apprehensive about structural stability of roof with any solution that did not call for removal and replacement of the slab.
  * Outline - Japan Engineer District Packet
    * 1. Daniel, Mann, Johnson, Mendenhall (DMJM) Report

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- a. Mike Phillips inspected proj Misawa 30 July 85. Discussions 30/31 July at res engr office.

- b. Field Inspection: Phillips/Harley Rowe/Dan Bruggenjohann

  - (1) Roof slab extensively cracked. Cracks over columns & along column lines similar to pattern obtained by sprinkling iron filings onto sheet of paper over poles of horseshoe magnet. At exterior corners, cracks are at 45 degrees. Along walls, cracks are parallel to wall, 3" apart, extending 2-3 feet into slab. Crack pattern appears to be perfect match to bending stresses.

  - (2) Cracks sharp edged (Ph 4) w/widths .5mm to .7mm(Ph 5 & 6). (.7mm = .027") Majority of cracks very tightly closed. Widest cracks (.7mm) occur around periphery of column caps.

  - (3) Three cored samples taken from slab; one adjacent to col cap (Ph 7, 8 & 9); 2d 3.5' from col, & 3rd close to center of bay in uncracked area. 1st & 2d core taken thru crack. Core inspection showed:

    - (a) Cracks do not propagate thru aggregates, but follow aggregate faces.

    - (b) Top rebar is 2-1/4" clear from top of slab in lieu of specified 3/4".

    - (c) As-built thickness of roof slab is 7-1/4" rather than 7" specified.

  - (4) No cracks visible on bottom of roof slab. Contractor notified DMJM he found cracks forming on underside. No location reported.

  - (5) Rowe/Bruggenjohann reported cracks still forming and growing.

  - (6) Field Inspection Records showed:

    - a. Concrete 210 kg/cm2 (3,000 psi) as specified.

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- b. Rebars SD 35 deformed, as specified.

- c.Column caps cast with slab, as they should be.

- d.Roof placed in two days: Morning slump 2 1/2"; afternoon: 3".

- (7) Cracks not noticed by POJNA inspectors until about 2 days after form stripping. Contractor has not said if he noticed cracks any sooner.

- (8) Walls/columns are not cracked.

- (9) Floor SOG was in place and used to support shores of roof formwork.

- (10) No deflection measurements taken when cracks discovered; nor have any been taken to date.

- c. 1st Discussion: 730 a-Phillips, Rowe, James Ditto (Dep Area Eng) Phillips comments follow:

  - (1) Cracks not due to punching shear which would have cracks larger than col cap, and failure would be immediate, not over time.

  - (2) Cracks that do not propagate thru aggregate are consistent with shrinkage or plastic settlement cracks, but pattern of cracking is not.

  - (3) Excess depth of top rebars decreases punching shear & moment capacity. Contributes to cracking, but not only cause.

  - (4)ACI manual covers load test procedure; fairly simple to perform;

  - (5) Cracks could have been early shrinkage that were not apparent until supporting form work removed. However, crack pattern more indicative of flexural cracks.

- d. 2d Disc 731 Discussions: Phillips, Rowe/Ditto/LTC

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LG Hassell (Northern AreaEngineer) Foll from HKM, DMJM, relayed by Phillips:

- (1) Even with excessive rebar cover, punching shear & moment capacities calculate as adequate for slab dead load.

- (2) Non-destructive testing such as impact hammers, pacometers could be used to verify f'c/depth, size and spacing of rebars.

- (3) Area of slab away from cols could be chipped to top of negative rebar to determine actual concrete cover depth, bar size and spacing.

- (4) Repair alternatives discussed. Steel beams under slab discussed and rejected. No acceptable alternative to replacement found.

- (5) Load testing discussed.

- e. Subsequent Data

  - (1) 1 Aug 85: Rowe informed Phillips core sample taken from roof tested 2,868 psi at 26 days.

  - (2) How much time between placement of wall, col and slab concrete; how much time between placement of slab concrete and curing materials. Rowe did not know.

  - (3) 6 Aug 85: Rowe informed DMJM water cured 28 day cylinders broke at 3,700 psi; job site cured at 4,050 psi.

- f. Analytical Investigation re Shop Drawings and Design Analysis

  - (1) Shop dwg rebar placement is in agreement with contract dwgs except:

    - a. Sheet A-44: Wall bars bent into roof slab shown in center of slab; cover dimension not indicated. Should have been placed in same layer as top parallel slab bars. Bent wall bars were intended to be contact lap spliced with top slab bars continuous at exterior slab

edge. Moment capacity at slab edge is marginal with wall bars at middle of slab.

- b. Sheet A-46: Bott bars are shown welded at col centerlines. Not req'd but OK since bott bars are unstressed here.

- (2) Des Analysis rechecked and found to be satisfactory.

- (3) (4) Additional analysis performed on conditions of exist roof slab.
  - (a) Data: f'c = 2800 psi; fy = 50,000 psi; slab t = 7.25"; cover = 2.75".
  - (b) Exist roof slab can safely carry its dead load plus an added dead load of 15 psf and live load of 15 psf. It cannot safely carry the mandated 40 psf live load.
  - (c) Punching shear capacity for full live load (40 psf) is marginal.
  - (d) Calculated max flex crack width is .47mm. At full live load, .75mm.

- (5) (6) (7) Calculated flex crack width based upon 90 % being less than value; however, isloated cracks may exceed twice the calculated values.
  - Expected flex crack width for DL: .25mm; under full load, .39mm.
  - Tolerable limit of crack width per ACI 224R-80 is .41mm.

- g. Conclusions
  - (1) Cracks are mainly flexural cracks associated with stresses in the rebars caused by self weight of roof.
  - (2) Although flex cracking major contributor, does not explain wide cracks. Probably no single cause; cumulative factors such as:
    - (a) Initial plastic shrinkage cracking

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occurring shortly after concrete placemnt. Shrinkage cracks may not be noticeable at midspan because they closed during bending of slab; ie, in compression zone.

- (b) Excessive rebar cover over top bars.

- (c) Flexural cracking occurring immediately after all forms were pulled, magnified by conditions "(a)" & "(b)" above. Forms should be in place as long as possible and then removed gradually in several stages over a period of time without shock or impact.

- (3) Even with top bars correctly placed and specified concrete strength attained, cracks still expected; however, widths would be about 1/2 the width of the observed cracks.

- h. Repair

- (1) Do Nothing: Load test must be conducted to determine adequacy of slab to carry des loads. If slab passes the test all large cracks should be repaired by epoxy injection.

- (2) Repair slab: Reshore roof, remove about 2" from top of slab, add rebars and replace w/epoxy concrete. After completion check by load test.

- (3) Demolish slab and rebuild: Completely remove entire roof including caps. Added bott & top rebars should be placed in first bay to take care of redistributed end moments, since it would be difficult to relocate exist wall rebars bent into slab.

- i. Photos 1 thru 12: At Interior Col Lines: Cracks radiating from Col; At corner, 45 degrees to walls; at edge, parallel to walls; crack shown in cored samples, skirting aggregate.

- 2. Q&A, Fai Seto Report

- (3) 26 Jul 85, Seto visited site. Foll Observed:

- a. PCF bldg 200' x 198' x 14' reinforced box type concrete structure, 12" sq int cols supporting roof loads & 8" thick exterior bearing walls transferring lateral roof diaphragm loads to foundations. Roof is flat slab system w/18" sq x 8" deep col cap.
- b. Slabs placed checker board pattern, 3-4 Jul 85.
- c. Special concrete strength tests made 17 Jul for form removal. Results from three field cured cylinders at 14 days showed f'c = 3642 average, exceeding 3000 psi contract reqmt. Contractor started form removal 18 Jul 85. Reshoring not seen during site visit.
- d. 20 Jul, Brueg observed irregular cracks on top of roof slab at every column. Cracks can be described as 19" diameter rings w/ lines extending radially outward from the ring's circumference. Crack widths varied from hairline to about 1/2mm. (.02"). Crack lengths varied from 12" to 48".
- e. Certain cracks still active, increasing in length.
- f. Brueg measured slab t thru roof vent hole near col as t = 7 1/4", exceeding the 7" t contract requirement.
- g. Top rebars over cols D13 grade SD 35 per specs.
- h. Maj rebars placed over cols appear to conform to contract plans. Shop Drawings did not show col cap rebar placement detail; d = 5 3/4" indicated. Unable to verify "d" used from photos or high chair samples.
- i. Wet foam backed plastic burlap curing material used for cont curing for 7 days. Surface cracks not observed during site visit 16 Jul when formwork still in place.
- j. Surfaces within ring shape crack appeared to be bulging upward 1/8"-1/4". Future ponding problem.

- k. Scaffoldings removed. No reshoring observed. No cracks observed on bottom surface of roof slab.
- l. No foundation settlement symptoms observed.

- (4) Discussions w/LTC Hassell/Ditto
  - a. Contractor believes roof cracking not his fault.
  - b. Contractor advised LTC Hassell he would do a site visit for info to establish probable causes of cracking and repair procedures.

- (5) FS requested roof slab cores.

- (6) FS rev AE cals; no obvious des errors.

- (7) 4 cores drilled by contr. Most surf cracks stopped at depth of top rebars except some cracks to middle of one core. "d" meas 4-1/2"; 4-5/8"; f'c 2860psi at 27 days.

- (8) Discussions
  - a.,b. Quality of concrete, rebars not considered major factors.
  - c. SD/Photos: Horiz rebar placement satisfactory.
  - d. Not considered punch shear failure, since no cracks on bott. Design Analysis indicate adequate provisions for punching shear. FS believes cracks caused by flexural strength failures at column strip areas.
  - e. 2870 psi at 27 days meet contr reqmts; Contractor asked to verify in-place concrete strength.
  - f.Comparing measured "d" = 4-5/8" w/des "d" = 5 3/4", 20% rebar shortage; inadequate rebars could jeopardize roof slab structural safety. Cracks caused by flexural strength failure due to low top rebar position and resultant inability to handle large negative moments over col cap area.

- (9) Recommendations

  - a. Result of contractor's negligence in selection of proper rebar support high chairs. Contr should submit proposal to resolve reduced roof load capacity instead of methods for repairing roof cracks.

  - b. Ponding check.

- (10) Photo showing rebars in place prior to concreting.

- 3. Contractor, Reiji Tanaka/Masahiro Kono

  - a. Contr requested des info.

  - b. Ltr Contractor 12 Aug(CHIZAKI KOGYO CO., LTD.)

    - (1) PCF Roof Slab structurally sound? Dr. Tanaka: Slab will remain in preset position for 1 or 2 yrs. After 2 yrs, will gradually deflect further. Serious concern during Seismic Activity; present slab very weak.

    - (2) Epoxy grouting: correct present cracking; in time further cracking most likely to occur. Different correction methods should be considered.

    - (3) Req des info; wants to meet des team; Contr to ask Tanaka for correction methods.

  - c. Tanaka/Kono Report

    - (1) Crack investigation. Job site visit Aug 7. Sketches 3.1-3.3 indicate pattern of cracking. Cracking around col range from .2mm-.6mm wide. Also, cracking around drop panel, probably due to punching shear/bending moment. Sketch 3.2 show typical plan view one grid; at mid point between cols, cracking occurred in straight line, considered bend cracking. Sketch 3.3 depicts cracking on underside; sketch 3.4 photos of actual cracking.

- (2) Design Conditions. Detailed calcs presented, w/conclusion: "Original Des is acceptable." Design of bending moment is considered correct around drop panel caused by punching shear. Critical section is at drop panel.
-
- (3) Punching Shear: as a result of crack investigation, we consider cracking around drop panel caused by punching shear,with critical section at drop panel.
-
- (4) Reason for cracks:
  -
  - a. Largest reason for cracking : puncaing shear around drop panel. Around drop panel rigid strength decreased thereby leading to radial cracks which is turned to bending cracks.
  -
  - b. Calcs presented. Conclusions: at time of form removal: cracks occurred
  -
  - c. If rebars correctly placed, cracking would not have occurred per calcs; once added loads placed, cracking would have occurred. Thus design is not considered to be preferable. Original des seems to be insufficient in considering the design against punching shear.
-
- (5) Conclusion: From investigation & discussions, we note following:
  -
  - (a) Large reason was cracking around drop panel was punching shear which decreased support strength around column and therefore concluded to additional cracks.
  -
  - (b) Design in terms of bending moment is adequate.
  -
  - (c) Design in terms of punching shear was not sufficient. Also it appears cracking due to long term loading was not considered.
  -
  - (d) In future, recommend:
    -
    - (1) This type design should be checked and reconsidered.
    -

- (2) Considering creep and long term loading and shear, added reinf should be added.
- (3) For similar buildings to be constructed, redesign should be considered.

- 4. End JED Packet Outline

- Research Items

  - 1.ACI Lessons from Failures of Concrete Structures

    - a. Categories of Failures

      - (1) Design Deficiencies
      - (2) Problems during construction, with special attention to formwork.
      - (3) Problems of durability and compatibility of materials.
      - (4) Problems re foundations & other particular types of structures

    - b. Notes on Flat Plates

      - (1) Flat plate floors are feasible and economical only if uninterrupted continuity at cols is provided. Openings at face of cols is prohibited.
      - (2) Temperature and Shrinkage Problems

        - (a) Nonuniform rates of shrinkage in elements of different thickness and shape causes rotational as well as lineal displacements. Coeff of shrinkage may be twice as high for a 6 inch thick vs a 12 inch thick concrete element.
        - (b) In addition to shrinkage which introduce large tensile stresses, there are stresses from temperature differences which compound the effect. If no reinf is provided across section, it takes only a few degrees temp change to result in stresses in excess of max tensile resistance of plain concrete.

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- (3) Construction Problems Leading to Failure
  - a. Inadequate over-all supervision and inspection.
  - b. Poor mixing and placing practices.
  - c. Cold and hot weather. In Hot Weather: Setting accelerated; strength reduced; tendency toward c:acking increased; adequate curing becomes mo-e critical.
  - d. Special Erection Problems
  - e. Formwork collapses
  - f. Premature Form Removal Has caused numerous failures; sagging, hairline cracks.

- c. Notes on crack Widths/Control-MCP 224R-18
  - (1) Steel stress most important variable.
  - (2) Thickness of concrete cover is important variable.
  - (3) Area of concrete surrounding ea rebar also important.
  - (4) Size of bottom crack width influenced by amount of strain gradient from level of steel to tension face of beam.
  - (5) 224R-18 formula 4.8 could be used for predicting max crack width in two way slabs and plates.
  - (6) 224R-21: Effects of long-term loading: can vary 10%/1000% over span of several years. Large scatter: indicate doubling of crack width with time can be expected.
  - (7) Plastic shrinkage cracks occur in surfaces of floors & slabs when ambient job conditions are so arid that moisture is removed from concrete surface faster than it is replaced by bleed water from below. As moisture is removed, surface concrete contracts, resulting in tensile stresses in the essentially strengthless, stiffening

plastic concrete, that cause short random cracks in the surface, usually only a few inches in depth. Range from few inches to a few feet in length, and are a few inches to two feet apart.

- Findings
  - a. Des Analysis OK.
  - b. Major discrepancy is rebar placement noted from measurement in field drilled cores. A loss of shear and moment capacity of 20% results from this factor.
  - c. Misplacement of wall bent rebars could be the cause of excessive cracking along the walls and at the corners. The possible ommission of specified corner rebars could also have contributed to the excessive cracking reported at the corners.
  - d. Slab rebar misplacement results in unacceptable reduction in shear and moment capacity.
  - e. Dr. Tanaka's punching shear calcs uses face of drop panel as critical section for punching shear; US practice & DMJM use d/2 from face. Dr. Tanaka calls thickened slab drop panel; DMJM refers this item as col cap. Dr. Tanaka states"not much allowance is given, thus, the design is not preferable." He also states drop panels are too small.
  - f. DMJM design does not use nor need drop panels; they use col cap instead. This is acceptable US practice. Dr. Tanaka states: For instance, in Japan, flat structures are calculated per the following recommendation· .41 for length of drop panel; ACI requires .33l which is not so different from Japanese practice. However, in DMJM's design, drop panels are not used and are not required. Col caps are used.
- Recommendations
  - Initial:
    - a. DMJM #1 only if contractor requests this. Analytical assessment indicate slab as built is structurally deficient, therefore we are not

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recommending load test.

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• b. DMJM #2: Remove top concrete/add rebars/concrete/ consider enlargement of capital at underside, provide analysis of fix by structural engineer. Load test after fix completed desirable. Recommend concrete removal to top of exist top rebars.

•

• 8/26/85, Phone Call, JED to POD

•

• JB: Contractor is asking for permission to demolish entire structure tomorrow. He will then build structure according to plan. JED wants to know if structure rebuilt according to plans will exhibit the same cracks. I responded that our review so far indicate DMJM's design meet ACI standards; if built according to plans, cracks of the magnitude noted in the first construction should not occur. Major cause for cracking is the misplaced steel. Other causes include the omission of bent wall rebars to the top of the slab, and the possible omission of corner rebars specified on the drawings. JB stated he may look into having Contractor discuss design with AE, and perhaps mutually agree to any improvements to the design.

•

• 8/27/85, Phone Call, JED to POD

•

• 1. Decision is firm to remove roof slab all the way to bott of cols.

•

• 2. Only data wanted from us is whether AE's Calcs are OK.

•

• Packet Rec'd from JED, 1300 8/30/85 2d Report & Letter From Contractor

•

• 1. DF: Ditto, Deputy Northern Area Engineer, to POJCD:

•

• Contr & their consultant indicate they still do not accept the present design as being sufficient. Request complete review of our design be made.

•

•

• 2. Contractor's Letter (Chizaki Kogyo Co., Ltd.)

•

• a. Corrective methods to three (3) choices:

•

• (1) Increase drop panel size.

- (2) Demolish exist roof slab and columns and reconstruct using larger drop panels.

- (3) Demolish existing roof slab and columns, and reconstruct same iaw contract drawings and specs. Contractor states for this alternative, they anticipate cracks will again occur but will eventually increase to structurally unacceptable sizes. We feel that in time, even with a rebuilt roof slab in strict accordance with the contract documents, same will become structurally dubious in terms of withstanding dead, live and most notably seismic loading.

- 3. Contractor's Consultant Tanaka "Investigative Report". 8/21/85

  - Summary of pertinent items this report. DMJM report correct. Main reason for cracking is lack of resistance against punching shear. Concept and calculation for punching shear is correct. Present rebar location correct for punching shear. Cracking is probably due to both bending and shear. d actual = 5.25":(previous submission showed d actual = 4 5/8"? Reason for difference unknown.) Consider main reason for cracking to be a combination of both bending and shear. Bending and shear usually checked separately. DMJM did not check for combination; depends on judgment of individual designer. To leave slab as is would be very dangerous due to earthquake loading & creep.

- 4. Basic Plan for Correction

  - a. Tanaka's plan for correction requires removal of existing col capital and the installation of a large drop panel. This no doubt would increase moment and shear capacity; however, the added expense may not be justified, since the original design does meet the standards of a nationally recognized code, and if rebars had been properly placed and all aspects of good workmanship observed, cracks of the magnitude noted would not have occurred. Mr. Tanaka in essence is saying that even if the ACI code is met, the design is not satisfactory.

  - b. Added strength must be added for bending stress and punching shear.

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8:10:48 AM 10/29/85 For Geo Matsumura, HQUSACE: Inquiry re Proj Ladylove:

a. FY84MCP, Ladylove, Perm Capability Facility, Misawa Air Base. Roof slab, rein conc flat plate with small column cap, cracked extensively after form removal. We were asked and reviewed calcs done by AE and found calcs satisfactory. Contractor on project elected to tear down entire concrete roof slab and at his own cost reconstructing roof structure using larger column capitals.

b. Resume of fonecall: GM stated Huntsville Engr asking him questions on design of Ladylove: Informed GM that the Chief of Tech Engrg Division, POD, visited the site; original structure has been demolished; contractor elected at no additional cost to government to rebuild with some modification at columns (enlarged caps). Informed GM design met ACI code. George asked about seismic; checks out with current codes. GM will let me know if he needs more info. No further action required.

Subject: DF, DAEN-ECZ-B, 15 Nov 85, Para 2.o.. re Roof Slab Failure at Misawa AFB.

Following is a synopsis of PODED-TS involvement in the subject item:

On 13 Aug 1985, PODED-T discussed subject matter w/John Ball(JB) and Ted Kishimori(TK) of JED. Main points of discussion were that the PCF building roof slab show many cracks upon form removal; rebars were not correctly placed; and a punching shear problem suspected. On 20 Aug 85, we received documents relating to roof slab failure. JB and Dave Dulong (DD) requested our comments relating to adequacy of the AE's design analysis.

Documents from JED included studies by the AE (DMJM), JED's QA Branch engineers, and data from engineers retained by the Contractors. We evaluated submitted documents and concluded that the design analysis prepared by the AE was satisfactory, and the major discrepancy was the rebar misplacement noted from field measurements.

On 27 Aug 85, DD of JED stated that decision had been made to remove roof slab and columns all the way to bottom of columns; JED wanted statement from POD whether AE's calcs are satisfactory. Letter to JED covering this sent 30 Aug 85.

In sum, PODED-TS involvement in subject item was to review the AE's design analysis and provide JED with our professional judgment on the adequacy of the AE's calculations. Our judgment was AE's calcs were satisfactory. To provide this judgment involved review of many documents. Detailed description of the investigation available (16 sheets attached).

PS: It is my understanding that roof structure has been rebuilt with some modification to the column capitals made by the Contractor at no additional cost to the Government.

2:05:17 PM 12/10/85 AS

Columbia River Fish Passage Structures

by
David C. Raisanen[1]

Abstract

This report covers the steel and concrete structures built at some of the mainstem Columbia River dams in the past 15 years to mitigate the passage of juvenile salmonids. As the reader is aware, the construction of dams on the river has placed barriers in the way of migrating species of fish, especially the salmon and steelhead. Facilities at these dams were designed by the Corps of Engineers initially to make sure that adult fish could make their way past these concrete obstacles to their spawning grounds. Not as much effort was expended to ensure the passage of juvenile fish past the dangers of the spillways and turbine blades on their way to the Pacific ocean. The great majority of the Corps' effort in fish facility design for the past 15 years has centered on how to ensure the survival of as many of these small fry as possible. Hydroelectric Design Center has been involved in the design and installation of many types of structures that help these juvenile fish in their struggle to reach the ocean.

Each of these structures has posed new kinds of problems for the structural engineer to solve, working together with mechanical engineers, fish biologists, construction personnel, and operating personnel. The result has been improved percentages of surviving juvenile salmonids, increased return of adults for the commercial and recreational fisherman, and preservation of a precious national resource.

General

The basic system that has evolved for juvenile fish bypass of the mainstem Columbia river dams centers on two main areas. The first was to modify the ogee shape of the spillways with a lip to flatten the flow, not allowing the water to plunge deep into the downstream area. This decreased the nitrogen supersaturation problem that causes fish kill by a process similar to the "bends" experienced by a diver who surfaces too rapidly. The other main concern was to prevent the young fish from entering the powerhouse intakes and being hurt or killed by the turbine action. The solution to this problem will be the main subject for this paper.

Traveling Fish Screens

Biologists have found that juvenile salmonids tend to drift downstream with the river flow on their journey to the sea. They usually face into the flow, so in actuality, they are drifting backwards. Since the great majority of the water in the river goes through the

[1] Hydroelectric Design Center, US Army Engineer Division, North Pacific; Portland, OR.

powerhouses, this means that they will too, unless some method to divert them is used. The powerhouses on the Columbia use a mechanically submerged traveling screen to divert them into the gate slot of the intake and a vertical barrier screen between slots to prevent them from returning to the intake passage. See Figure 1 for a cross-sectional view of a typical system.

through the dams. Early testing was done on the dams on the Snake river, such as Lower Monumental and Lower Granite. The first traveling fish screen and vertical barrier screens were installed at Lower Granite Dam in 1975. A short explanation of these two structures would probably be helpful at this point. A traveling fish screen is a steel framed structure that has two rotating polyester screens, much like

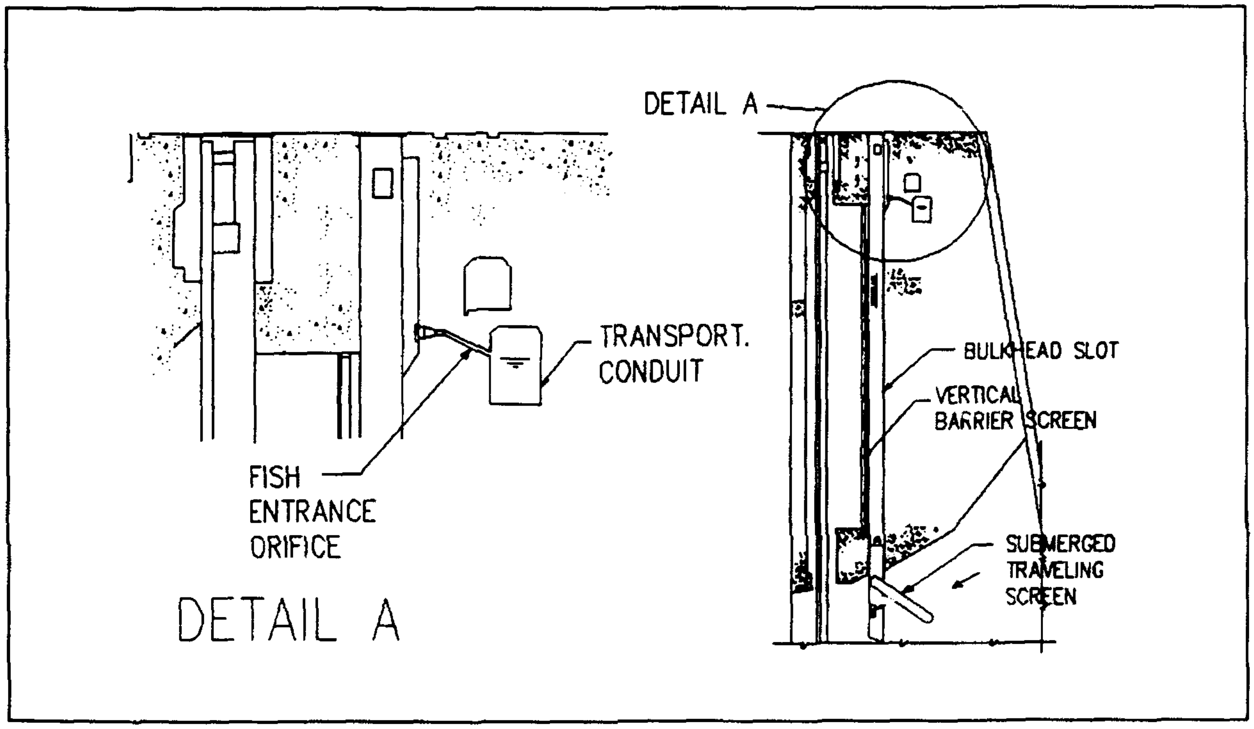


Figure 1. Cross section of intake

From the gate slot, they flow the flow to an entrance orifice which leads to a concrete transportation channel. This channel is open channel flow and it goes the full length of the powerhouse intake. It will lead to a fish ladder or to a juvenile fish outfall pipe. These both empty into the powerhouse tailrace, where the fingerlings can continue their downstream journey.

None of the dams on the Columbia river were designed with such a system, so all had to be retrofitted with the necessary concrete and steel structures. The Corps and the National Marine Fisheries Service (NMFS) cooperated in conducting a series of testing in the 1970's and 1980's to determine a method for improving the efficiency of our fish passages

wide conveyor belts made from polyester mesh, mounted on a hinged inner frame. It is installed in the intake water passage by lowering it down the intake bulkhead slot in the flat position, and then the outer frame is dogged into position so the inner frame can be swiveled out at a 55-deg angle into the flow. The outer frame rides on the bulkhead guides in the gate slots. Since testing showed that the fish are located in the upper portion of the water passages, a 22-ft inner frame length was used for all of the Columbia river powerhouses. The intake piers are 20 ft apart on most projects, so this dictated the width of the frames. As the fish reach the traveling screen, they are gently forced into the bulkhead slot. The vertical barrier screens are installed in the area

above the gatewell beam between the intake gate slot and the bulkhead slot. This screen prevents them from returning to the intake, so they move where the water takes them, i.e., through the entrance orifices. Much testing was done by NMFS personnel to determine all of the settings for the equipment, such as the positional relationship between the gatewell beam and the traveling screen, the degree of porosity required for both screens, the best angle for the traveling screen, and the polyester belt travel speed. Design loading for these screens is presently a uniform load of 180 psf, which corresponds to a plugged screen condition. The allowable stress for this condition is 45 percent of yield point. We also analyze the screens for vibrational modes to make sure their natural frequencies do not approach the ones found in the intake. A picture of a typical traveling fish screen showing how it appears prior to placing it in the slot is shown in Figure 2.

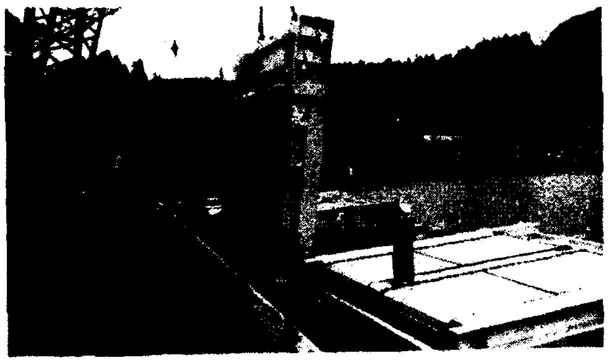


Figure 2. Traveling fish screen

The vertical barrier screens are typically made of 8-ft, 6-in.-high sections with a 19-ft span. The steel framework is A-36 steel. The present design load on these screens is a uniform head differential of 2 ft, which corresponds to a plugged screen. The allowable stress for this loading is 50 percent of yield point. They fit into A-36 steel guides that were installed by grouting anchor bolts into the existing concrete at 8-ft centers. The guides are built of wide flange sections with a "c"-shaped guide welded to them. After adjustment, the guides are grouted in place. At the present time, the screens are installed or removed from intake deck level one panel at a time. They are lowered into the bulkhead slot and swung downstream to engage the guides. Future plans call for removal of a chunk of the intake deck concrete to form an access directly above the screens so they can be lifted straight up. The screens will also be bolted together so they can be lifted as a unit.

Transportation Channels

One of the big problems confronting us was how to form a channel to transport the fingerlings once we got them through the gate slot orifices. One way was to convert part of the ice and trash sluice into a transportation channel while still retaining an operational sluiceway. This approach was used at McNary, Bonneville, and The Dalles, which all had the requisite sluice. Figure 3 shows a cross section of the retrofitted fish channel at the Bonneville first powerhouse.

This channel was designed to take the impact of 3-ft-diam, 20-ft-long logs, approaching at a velocity of 6 ft per sec at right angles to the wall. It is also designed for the water forces with the channel empty and the sluiceway operating, or the channel operating with the sluiceway empty.

At Bonneville, the juveniles enter a fish evaluation structure upon leaving the transportation channel, where NMFS personnel can inspect them, or mark them as desired. They then enter an outfall pipe that releases them below the surface of the tailrace.

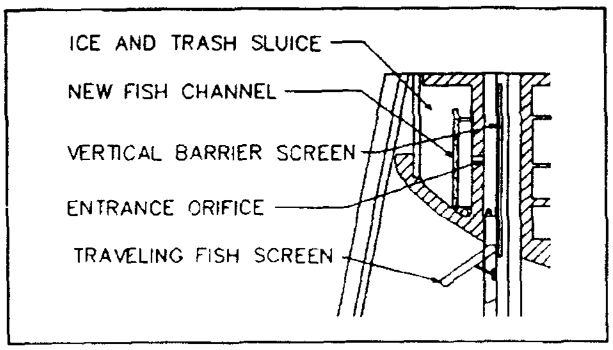


Figure 3. Bonneville fish transportation channel

Some of our powerhouse intakes, such as the one at John Day, did not have an ice and trash chute, so it became necessary to mine a new conduit into the mass concrete of the intake structure. This powerhouse has 16 generator bays plus an erection bay and station service bay. The total length for the new transportation channel tunnel is 1,600 ft. The finished cross section is a maximum of 8 ft, 3 in. by 17 ft, 0 in. at bay 1 and decreases in area toward bay 16. At each generator bay, the channel is connected to the bulkhead slots by a mined orifice gate opening. After mining, the channel was given an 8-in. concrete cover on all sides to provide smooth walls, which prevents descaling of the juvenile fish. Figure 4 shows a plan view of the transportation channel through a typical generator bay.

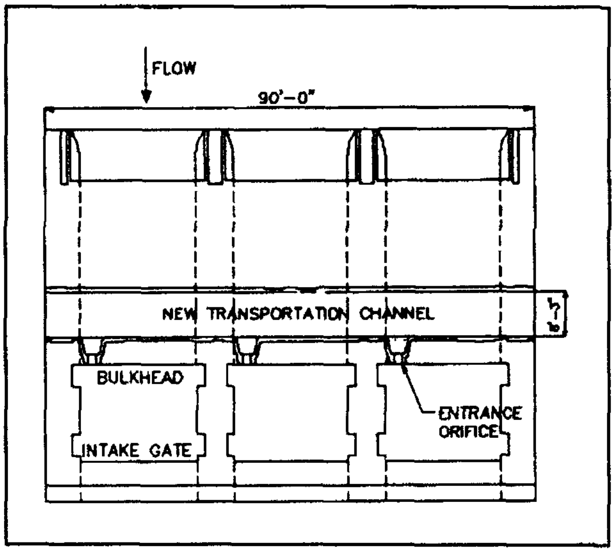


Figure 4. John Day generator bay intake

The mining method was left to the contractor and the specs gave the concrete strength and aggregate size. The contractor chose to do the mining with a Roadheader machine, a machine that has its cutting head mounted on a hydraulic boom. The head has conical teeth and rotates rapidly to cut through material. It was tested on an abandoned bridge abutment. Unfortunately, the choice was not a good one, and his progress was very slow. When phase one of the job (nine generator bays, erection bay, and station service) was finished, the Roadheader was also finished. The contractor was allowed to use a circular tunnel for the remainder of the units and used a horizontal boring machine. Litigation, of course, was used by the contractor to try to recoup his losses on the first phase of construction. His claim was that the Corps was remiss in not correctly specifying the hardness of the existing concrete.

Operation of the system is simple in theory. The water flows from generator bay 16 toward bay 1, and as it does, the cross-sectional area of the channel increases, because each orifice is adding flow to the system. When it exits the erection bay, the mined channel makes a turn toward downstream, going through a nonoverflow monolith. It then turns into a formed concrete channel that leads down to a fish handling facility situated on the left bank, where fisheries personnel can inspect, mark, sort the juvenile salmonids, and return them to the river. A small tainter gate located in the concrete channel, just downstream of the nonoverflow monolith and in conjunction with the orifice gates in each gate slot, controls the flow rate in the system. This project uses the standard length submerged traveling screens (STS) and vertical barrier screens in each gate slot.

Turbine Intake Extensions

Bonneville's second powerhouse, completed in 1981, posed a new type of problem for juvenile fish passage. It was designed with a shorter upstream-downstream length of intake than the older powerhouses on the river. This was done for two reasons. First, an ice and trash sluiceway was eliminated, and secondly, a shorter intake deck was needed. This shortened the waterway roof from the trashrack to the bulkhead slot, and it became a sloping straight line instead of a curved surface, as in older structures. It was fitted with standard-length STS and put into operation. A cross section of the intake structure, including an outline of the Turbine Intake Extension (T.I.E.), is shown in Figure 5.

Testing by NMFS showed that the fish guidance efficiency of this arrangement was much lower than previous plants. Hydraulic model testing of the intake revealed that the

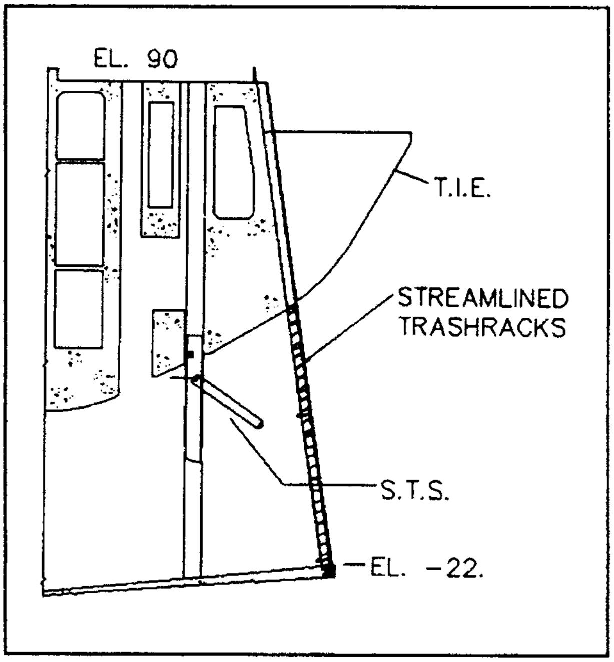


Figure 5. Bonneville second powerhouse intake

flow characteristics of this intake were such that a large number of the juvenile fish were being routed beneath the screens. A solution to this was to return to the gradual curve for the waterway roof used in older plants. We did not wish to rebuild the intake with concrete, so we designed a steel T.I.E. that slid down the existing trashrack guides and rested on the trashracks. The T.I.E. measures 34 ft by 26 ft in plan and is 40 ft high. It weighs 62,000 lb. We also designed new streamlined trashracks that had horizontal vanes to modify the flow pattern. A cross-sectional view of the T.I.E. is shown in Figure 6.

The skin plate on the upstream side of the T.I.E. is 3/16 in., grade 50 plate, and the shape was picked to approximate the shape of the intake roof at McNary powerhouse, which has a high fish guidance efficiency. The T.I.E.'s were designed to withstand both longitudinal and lateral currents, wave loadings, ice loads, wind loads, and impact due to trash. The prototypes were not designed for seismic events, since they were scheduled to be used for one season only, but have since been upgraded, and are currently in their sixth season. Fish guidance efficiency has improved because of the T.I.E. usage but still needs improvement. We had to strengthen the trashrack guides for T.I.E. reactions in the upstream direction. This was done by installation of steel plates anchored to the intake pier noses by hollow core groutable rock anchors. The plates bear against the upstream surface of the existing guide steel. The T.I.E's are installed and removed by the use of large barge-mounted cranes, which is a high-cost item, so studies are now being completed to come up with a T.I.E. that is either permanent or easily installed by Corps personnel. Figure 7 shows the T.I.E.'s installed at the Bonneville second powerhouse.

Long Submerged Traveling Screens

Another idea to increase juvenile fish guidance just now being tried is to lengthen the STS length from 22 ft to 40 ft. This will increase the percentage of the waterway area covered by the screens and increase the number

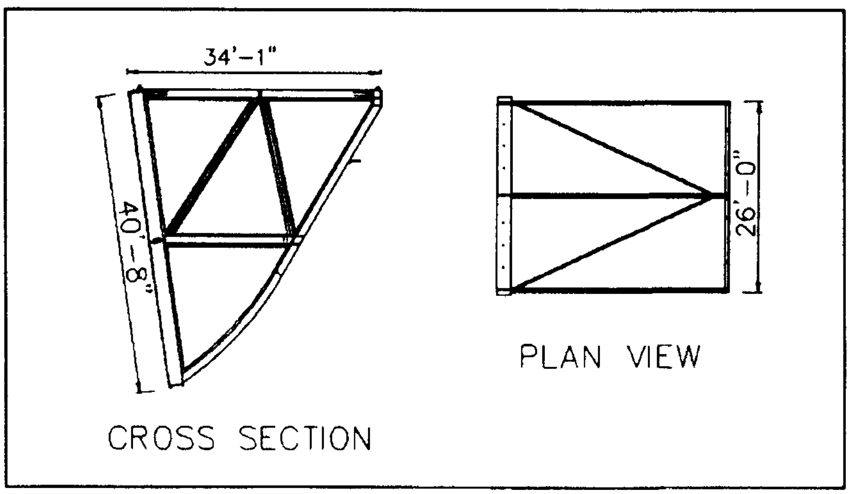


Figure 6. Turbine intake extension

intercepted. The original 22-ft screens are lowered into the slot, dogged off, and the inner screen with the mounted revolving mesh belts is pushed into its operating position by two arms attached to a slider mechanism on the outer frame and 5 ft from the end of the inner frame on the other end. When the inner screen length is increased to 40 ft, the number of arms required for support increases to four. The main reason for this was to keep the submerged natural frequency of the structure above 10 Hz. This was done to make sure resonance did not occur during operation of the screens. All static and dynamic analysis was done using the program STAAD-3 (Research Engineers, Inc., latest version). Because of the complex procedure required to raise the inner frame, we had to be sure of its feasibility early in the design, so we used AUTOCAD's (Autodesk, Inc., latest version) 3-D features together with AUTOFLIX (Autodesk, Inc., latest version) to produce an animated 3-D model of the STS on our computer monitor. We used this model to check parts movement and interference. Due to this early work, when the prototype structure was run through its testing procedure, everything worked as planned.

Figure 7. Turbine intake extension installation

This was a good example of mechanical and structural engineers working together to produce a product required by biologists and environmentalists.

Figure 8 shows an elevation view of the new STS, and Figure 9 shows a photo of the prototype structure in the fabrication yard during -

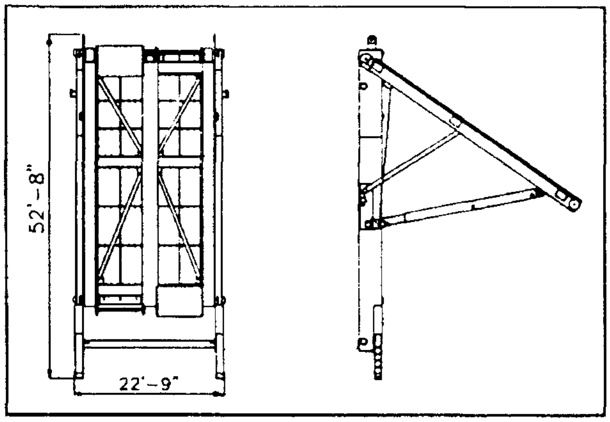


Figure 8. 40-ft STS

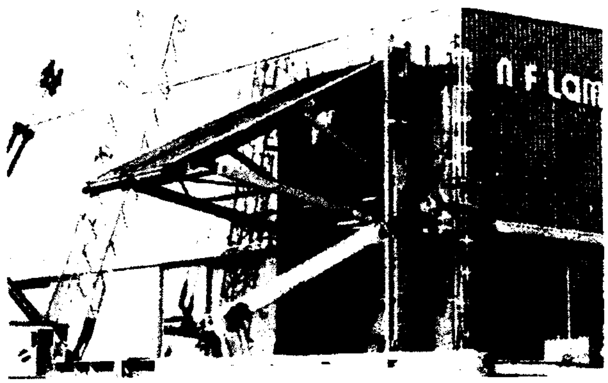


Figure 9. Prototype STS during testing

initial testing. This prototype is presently being tested by NMFS personnel at the McNary powerhouse. Structurally and mechanically, it is performing as expected, but final fish guidance results are not known at the present time. The new 40-ft STS design is also planned for The Dalles and Lower Granite projects.

Another idea we are looking into is raising the position of the intake gates in their slots, by 10 to 15 ft, to improve the flow of water over the STS and into the bulkhead slot. Raising the gate increases the flow area in the intake gate slot leading back to the turbine waterway. Since project personnel frown on the idea of having gates or hydraulic hoists sticking above deck level, we will have to modify the gates and hoists by moving the attachment point down near the bottom of the gate and increasing the length of the hydraulic cylinders. This will be tried at McNary also.

Conclusions

The installation of these structures has increased the chances for juvenile fish passage. We use the term "fish guidance efficiency" to evaluate our results. This number represents the percentage of juvenile fish coming down the river to the powerhouse that are successfully guided into the gate slot. Another term used often is "dam passage survival." This is a percentage of juveniles that successfully go through each project. The fish guidance efficiency for each project varies, as does the percentage for each species of fish, so it is very difficult to assign one number to the overall success of these methods. Figure 10 shows existing and improved level results for the five Columbia River powerhouses.

The existing condition was as of 1985 and includes improvements made to that point. Improved level numbers include long STS, gate raises, bypasses at all projects, and Turbine Roof Extensions at Bonneville 2nd Powerhouse (US Army Engineer Division, North Pacific, 1988). Results for the new 40-ft-long STS show a FGE of 85 to 93 percent at McNary, but these are highly preliminary.

According to my own calculations, the construction cost for these structures at the Columbia river and Snake river projects is approximately $120 million to date. This does not include E&D costs, testing, and many miscellaneous expenses. We expect to spend another $170 million more in the next 10 years on new methods and improvement of existing facilities. It becomes evident that the Corps and the National Marine Fisheries Service are making a major effort to ensure the preservation of this national resource. Recent studies in the Pacific Northwest have shown declining numbers of several species of salmon in our rivers, and the Corps of Engineers has taken much criticism for a perceived lack of interest in preserving these runs. It is my hope that this paper shows that the Corps is deeply involved in the fight to save the fish runs on our rivers, and that we will increase our efforts in the future to preserve and protect this national resource.

	Fish Guidance Efficiency, percent		Dam Passage Survival, percent	
Project	Yearling	Subyearling	Yearling	Subyearling
Existing Conditions				
McNary	75	40	95.6	91.2
John Day	72	30	94.4	89.1
The Dalles	40	40	90.8	90.9
Bonn. 1	76	72	95.6	95.1
Bonn. 2	19	24	87.7	88.4
Improved Level				
McNary	90	60	97.4	93.8
John Day	90	60	96.7	92.9
The Dalles	80	63	96.3	94.0
Bonn. 1	76	72	95.6	95.1
Bonn. 2	65	24	94.1	88.4

Figure 10. Existing and improved results

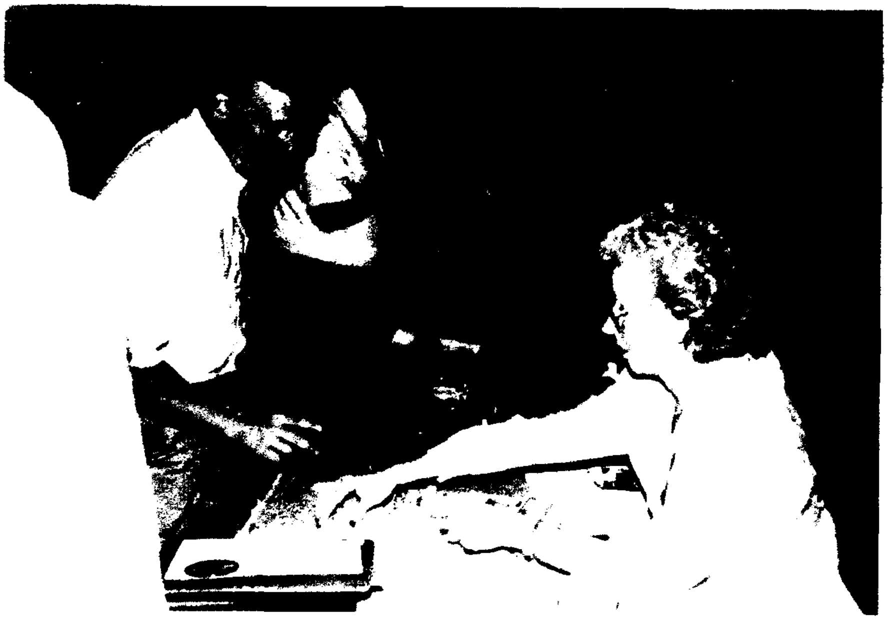
References

STAAD-III is a registered trademark of Research Engineers, Inc., 540 Lippincott Drive, Marlton, N.J. 08053.

AUTOCAD is a registered trademark of Autodesk, Inc., 2320 Marinship Way, Sausalito, CA 94965.

AUTOFLIX is a registered trademark of Autodesk, Inc.

US Army Engineer Division, North Pacific. 1988 (Apr). "Juvenile Fish Bypass Goals," Portland, OR.



The North Fork Toutle River Fish Collection Facility Design and Construction

by
Bruce H. McCracken, PE[1]

Abstract

The Portland District is completing construction of the $8 million fish collection facility (FCF) on the North Fork Toutle River, Washington, in the summer of 1991. The facility mitigates the fishery impacts of a 210-ft-high dam known as the sediment retention structure (SRS) constructed directly upstream of the FCF. Run by the Washington State Department of Fisheries, the facility is designed to trap and collect upstream migrating salmon. The trapped fish are trucked and released at natural spawning grounds, fish hatcheries, or downstream for recapture by sport fishermen.

History

On May 18, 1980, Mount St. Helens erupted. The eruption and accompanying debris avalanche deposited 3 billion cu yd of silt, sand, and gravel in the North Fork Toutle River Valley. Carried by a massive mud flow, the material filled the river banks of the Toutle and Cowlitz rivers and blocked 12 miles of navigation channel on the Columbia River.

The rivers were dredged and navigation was restored, but 550 million cu yd of sediment were predicted to erode from the avalanche during the next 50 years. To reduce future dredging and its costs, the US Army Corps of Engineers built the SRS. The 210-ft-high SRS automatically traps and stores sand and gravel that would have to b dredged out of the Columbia River navigation channel. Unfortunately the SRS stops all fish from migrating to any upstream spawning grounds.

The FCF mitigates the dam that blocks upstream passage of migrating salmon. In 1986, the Department of the Army and the state of Washington agreed that the Corps would construct the facility and the state would receive ownership, operate, and maintain the facility.

Features

General

The prominent features of the facility include a (1) fish barrier, (2) overflow spillway, (3) fish ladder, (4) collection pool, (5) fish lock, (6) truck loading area, and (7) water system. A photograph of the completed facility is shown in Figure 1. A general plan is shown in Figure 2, and a more detailed plan is shown in Figure 3.

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Figure 1. Completed fish collection facility

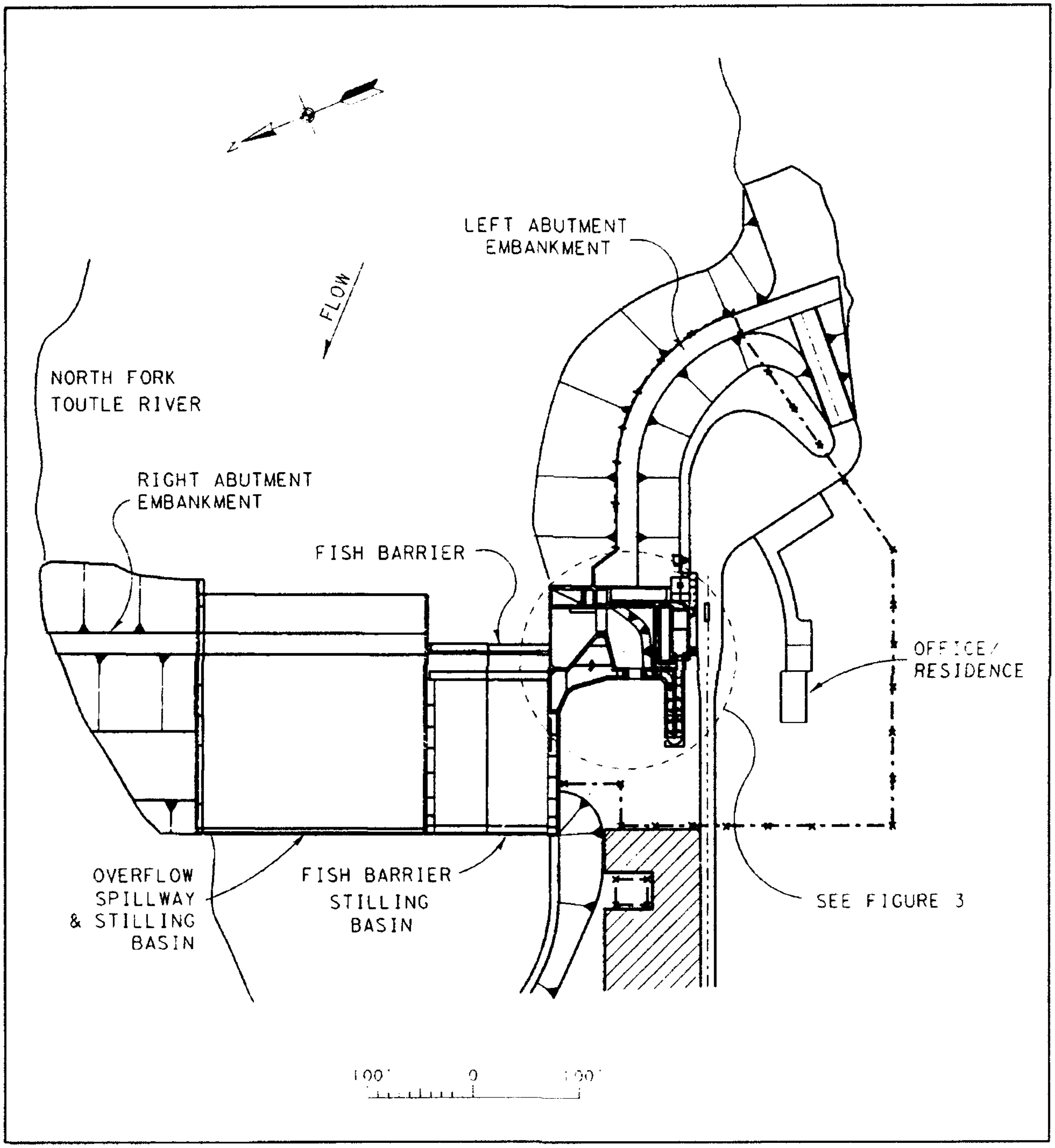

Figure 2. Fish collection facility general plan

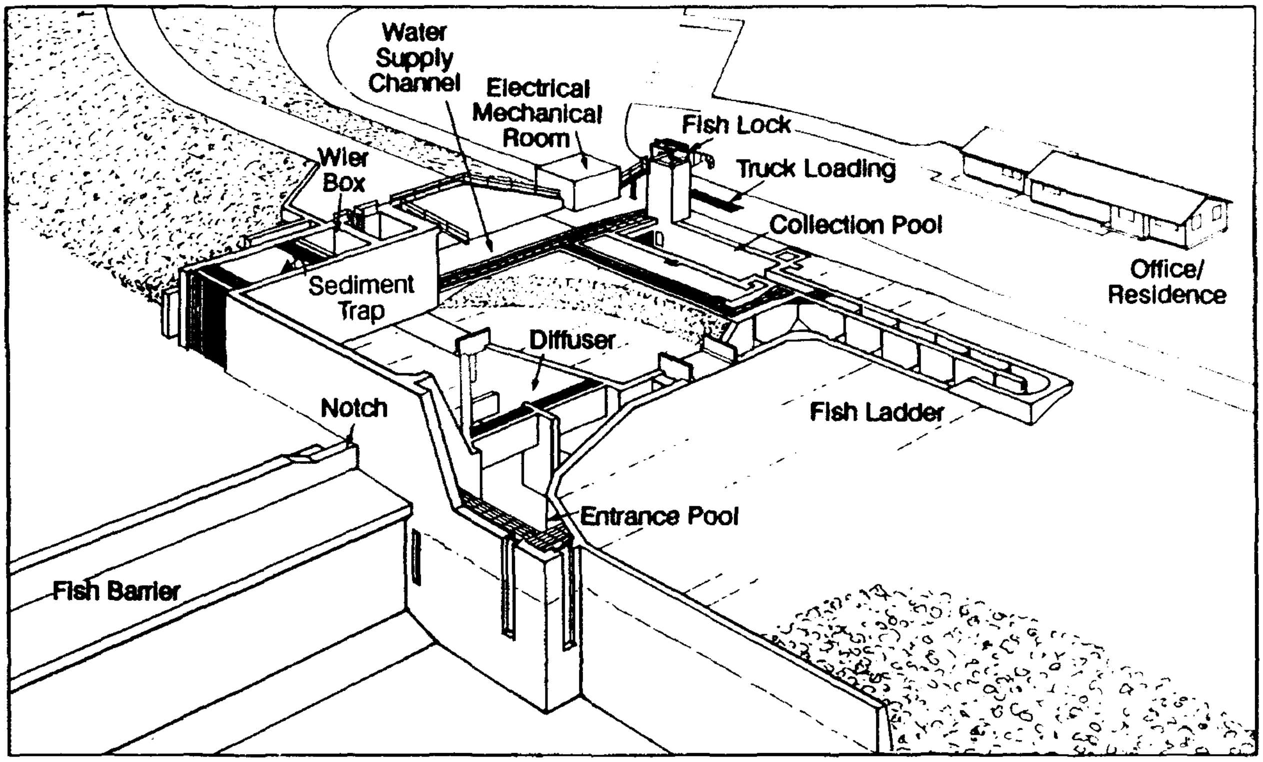

Figure 3. Fish collection facility detailed plan

Fish barrier and overflow spillway

The fish barrier is a 31-ft-high concrete dam that blocks the fish from swimming upstream of the facility. The barrier is 37 ft wide and 250 ft across. Downstream of the fish barrier is a 3 1/2-ft-thick stilling basin slab that is 115 ft wide. Adjacent to the fish barrier is the 233-ft-long by 215-ft-wide, 28-ft-high concrete overflow dam. The overflow spillway is a 4-ft concrete slab on soil foundation separated from the fish barrier stilling basin by a cantilever retaining wall. Both the fish barrier and overflow spillway slabs are anchored to the ground by prestressed, posttensioned soil anchors.

Fish ladder

The fish ladder is concrete and shaped like a narrow "U." It is 6 ft wide, 210 ft long, and rises 12 ft from the entrance pool to the collection pool. The weirs are removable to make sediment cleaning easier and to compensate for the future channel degradation.

Collection pool

The collection pool is 14 by 40 ft with 9-ft-high walls. It is sized to hold 450 salmon weighing as much as 10 lb. When the facility is being operated, a screen made of perforated plate, 7-1/2 ft high by 14 ft wide, lowers into the downstream end of the pool. An electric motor drives the screen upstream and crowds the fish into the runway. In the upstream end of the collection pool the runway is bounded by the upstream concrete wall, the pond crowder in its furthest upstream position, a runway crowder, and the fish lock entrance. The 7-1/2-ft-high runway crowder screen runs perpendicular to the collection pool crowder and forces the fish into the fish lock.

Fish lock and truck loading

Once the fish are crowded into the lock, the entrance is gated shut. The 4-ft by 6-ft lock fills with water to a level 16 ft above the collection pool. An electric hoist raises a perforated metal plate (called a brail) from the

floor to the top of the lock. At the top the brail slopes into a fish chute that funnels the fish into tank trucks.

Water supply system

The gravity-fed water supply system begins immediately upstream of the fish barrier where it passes through the sediment trap; drops over a weir into the weir box; exits into supply channels; and diffuses into the collection pool, fish ladder, and entrance pool. The sediment trap draws the water immediately upstream of the fish barrier dam. Sediment drains into a 1-1/2-ft-diam pipe and flows back into the river. Trashracks with 3-in. clear spacing keep the large pieces of floating debris out of the sediment sluice. From the trap, the water flows over a 15-ft weir into the weir box, through control gates, and outside to the supply channels. The supply channels direct flow into three concrete diffusers. The main diffuser supplies attraction water to the entrance pool, the second diffuses attraction water to the upstream end of the collection pool, and the third diffuses attraction water to the upstream end of the fish ladder. Steel bars spaced at 1 in. on center cover all diffusers and openings accessible to the upstream migrant fish.

Design

Design requirements

We based the design of the facility on operational and structural requirements. Operationally, the facility collects fish at discharges of 170 to 5,000 cubic feet per second (ft^3/s). Structurally, the facility resists forces associated with a 100-year flood (30,800 ft^3/s), normal static forces, and earthquake loadings developed by using a seismic coefficient of 0.20. Although seismic zone maps show a coefficient of 0.15 at the FCF, we chose to design the structures with the higher coefficient because of the close proximity of the FCF to Mount St. Helens and its recent seismic and volcanic events.

Sediment considerations

The SRS collects and stores sediment that would normally travel downstream to the FCF. As the SRS fills with sediment, increasing amounts of sediment will pass by the SRS and transport to the FCF. Eventually, the SRS will fill with sediment and the river will transport large volumes of sediment past the FCF on a continuous basis. To help keep the water supply intakes clear, we designed the fish barrier with a notch. The notch directs the flow past the entrance and cleans out the sediment. During review of the plans and specifications, the fish agency engineers felt turbulent flow downstream of the notch would confuse the fish, so we added a concrete trainer wall to guide the fish into the entrance pool. The agencies were still concerned about the turbulence, so they asked us to fill in the notch. After considerable discussion and letter-writing, we modified the contract to bolt wide flange stop log guides to the fish barrier and install removable concrete stop logs. Once installed, the guides caught floating debris and caused additional sediment to enter the facility, adding to the maintenance of the project.

Site selection

We designed all or part of the FCF on three separate sites and two interim sites on the Toutle River. The original plans in 1985 located the project downstream of the SRS outlet works exit channel and upstream of the SRS spillway outlet. Flows at the FCF would have been limited to 6,000-ft^3/s, since the higher spillway flows would be routed around the facility. For construction economy, the location made sense, but fish-wise it would not have worked. Normal, high, and completed project flow conditions were undesirable for one reason or another. During normal flows the velocity and turbulence exceeded hydraulic requirements. When flows are high, the water would go out of the SRS, down the spillway, and falsely attract the migrating adult salmon away from the facility. Finally, when the SRS fills up, no water would flow through the SRS outlet works to the FCF.

In April 1986, we moved the proposed site 4,700 ft downstream of the SRS. The new site was clearly out of the influence of the spillway, but was an enormous change in scope. Originally limited to about a 6,000-ft^3/s maximum flow, the new site would be subjected to as much as 213,000 ft^3/s during the probable maximum flood event. Equally important was the change from the rock foundation to saturated sediment foundation. Concurrently we completed the design of the SRS and began advertising the SRS construction contract.

In December 1986, the Corps awarded the contract for the SRS. Soon afterward, the SRS construction contractor decided not to share the access roads with the future FCF contractor. Construction and operation of the FCF during construction of the SRS was expected to generate traffic averaging one fish-hauling truck per day in addition to future FCF contractor traffic. After long discussions, the contractor convinced the Corps that because of traffic-handling problems, access to the new site was unsafe, and the site should be moved.

In May 1987, we moved the site another 3,800 ft downstream for its final siting. Much of the design from the last location was salvageable, but considerable time was lost. This action was not good for interim mitigation efforts. Ideally, the FCF would have been built before any work was done at the SRS. Instead, the FCF was not even designed. It was obvious that the SRS work was not going to wait for the final design of the FCF, so the agencies required some sort of a trapping facility for the fish to mitigate the stream blockage on an interim basis. The Corps initiated a crash program at the cost of $180,000 to build an interim facility to the Federal and state fishery agencies' specifications. A sediment trap upstream of the interim facility was designed to remove sediment before it reached the facility. Unfortunately, the trap was estimated to cost more than the interim facility and was never funded. When the interim facility was put into action, it became plugged with sediment and floating pumice and failed in its first day of service.

Still determined to trap fish, the agencies requested another interim facility. The second interim FCF would be as far upstream as possible and would get its water directly from the outlet works exit channel. We initiated another crash program to build a facility to trap on an interim basis. A key element in the construction process was to install a water supply pipe in the outlet channel before the SRS was diverted in the fall of 1987.

In a nonnegotiated change order, the Corps directed the SRS contractor to buy water pipe and blast a channel into the rock. Eventually, the cost for this work came to $72,000. Meanwhile, the cost for the new interim facility grew to $1,000,000.

In November 1987, the agencies and the Corps realized the interim facility was too expensive. The Corps stopped construction of the facility and agreed to provide the agencies with approximately $425,000 to restock the river. That winter season, the agencies hired sport fishermen to fish for steelhead salmon with barbless hooks for interim collection. However, the fishermen caught very few fish. Later in the season, flows eroded the rock in the outlet channel where the pipe channel had been excavated. In 1988, the erosion was repaired by backfilling with approximately 1,000 cu yd of concrete at a cost of $174,000.

Foundation and backfill

All of the foundations are built on saturated posteruption sediment, preeruption alluvial, or waste fill. Generally, the soil is cohesionless, weighs 130 lb/ft, and has an internal friction angle of 30 deg. Backfill loads were computed with an internal friction angle of 35 deg.

Structural analysis and the fish barrier

Structural analysis considered sliding, overturning, and flotation abilities. Due to the saturated soil and low friction angles, sliding stability controlled the fish barrier design.

An example of the fish barrier design criteria and computed factor of safety is shown in Table 1. Figures 4 and 5 show the computed loads and resultants of the worst case overturning stability and sliding stability analysis of the fish barrier and stilling basin. We analyzed the retaining wall overturning stability with static methods and calculated sliding stability with the CASE program CSLIDE. Uplift forces were calculated using the line of creep method.

Design of other structures was controlled by flotation stability as well as sliding. The unwatered condition in saturated soil tended to lift the structures from their intended locations.

Posttensioned slab analysis

The fish barrier stilling basin and overflow spillway slabs are anchored against uplift by 1-in.-diam, American Society for Testing and Materials Standard A 722, Grade 150 prestressed soil anchors. The fish barrier slab is anchored on the upstream end by three rows of anchors spaced at 9 ft parallel and 12 ft perpendicular to the streamflow. The overflow spillway slab is anchored by six rows of anchors spaced at 13 ft parallel and 10 ft perpendicular to the streamflow. The first row in the overflow spillway begins 57 ft downstream of the crest. All anchors in the overflow spillway slab were placed perpendicular to the 1V on 1H sloping face.

Design loads varied from 50 to 60 kips per anchor; lengths were typically 40 ft. We installed 96 anchors in the overflow spillway and 27 anchors in the fish barrier stilling basin. Costs for the anchors were approximately $1,800 per anchor for a total of $220,000.

Construction

In June 1988, the Corps awarded the contract to build the FCF to Humphries Construction, Inc., Woodinville, WA. This company completed the work in July 1990.

Dewatering

Keeping the water out of the site during construction was one of the major considerations in building the FCF. We planned on placing the main facility on a sand and gravel foundation below the water table in the existing river channel. To further complicate matters, the future river channel was expected to degrade or lower 6 ft below the existing elevation. This expectation meant that all of the foundations and structures had to be designed and placed deeper to enable the FCF to collect the fish as the channel degraded.

Of special interest was the closeness of the site to the river and the tight schedule for dewatering. Dry working conditions for a 10-year flood event (34 ft below the normal river level) during a winter construction season were required. A $1,100,000 dewatering system that included cofferdams, wells, and pumps did the outstanding job of keeping the river out. We designed six interconnected wells and six pumps to keep the water table low. We shotcreted the cofferdams on the water side to provide extra protection. The system worked extremely well.

The entire site was dewatered in 5 days. Three pumps, set at low output levels, kept the water table 10 ft below the excavation on nearly all but the rainiest days.

Table 1
Fish Barrier Stability Requirements and Results

	Sliding Factor of Safety		Minimum Base Area in Compression (Overturning)		Minimum Bearing Capacity Factor of Safety	
Loading Condition	Required	Computed	Required	Computed	Required	Computed
Usual	1.5	2.0	100%	100%	3.0	7.2
100-Year Flood	1.33	1.7	75%	100%	2.0	2.0
Earthquake	1.1	1.2	RWB	80%	N/A	N/A

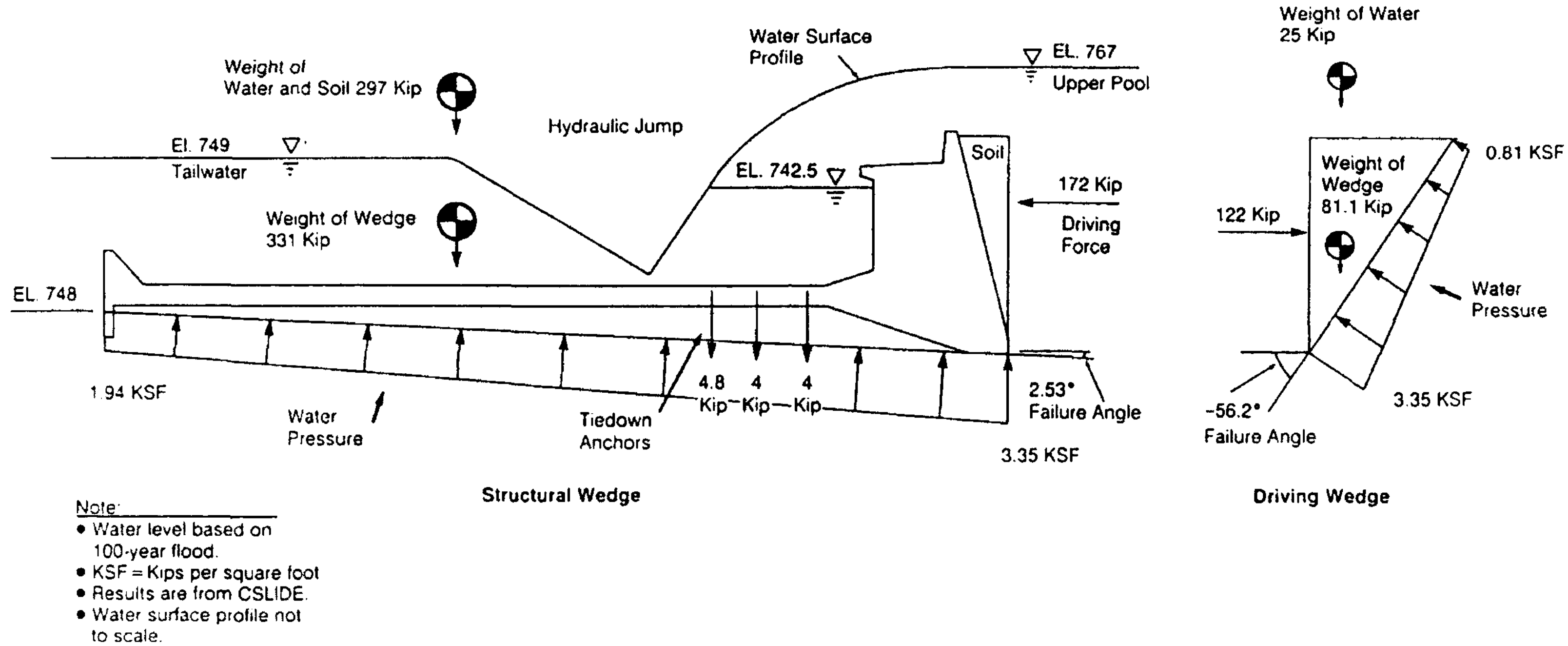

Figure 4. Fish barrier sliding stability extreme loading condition

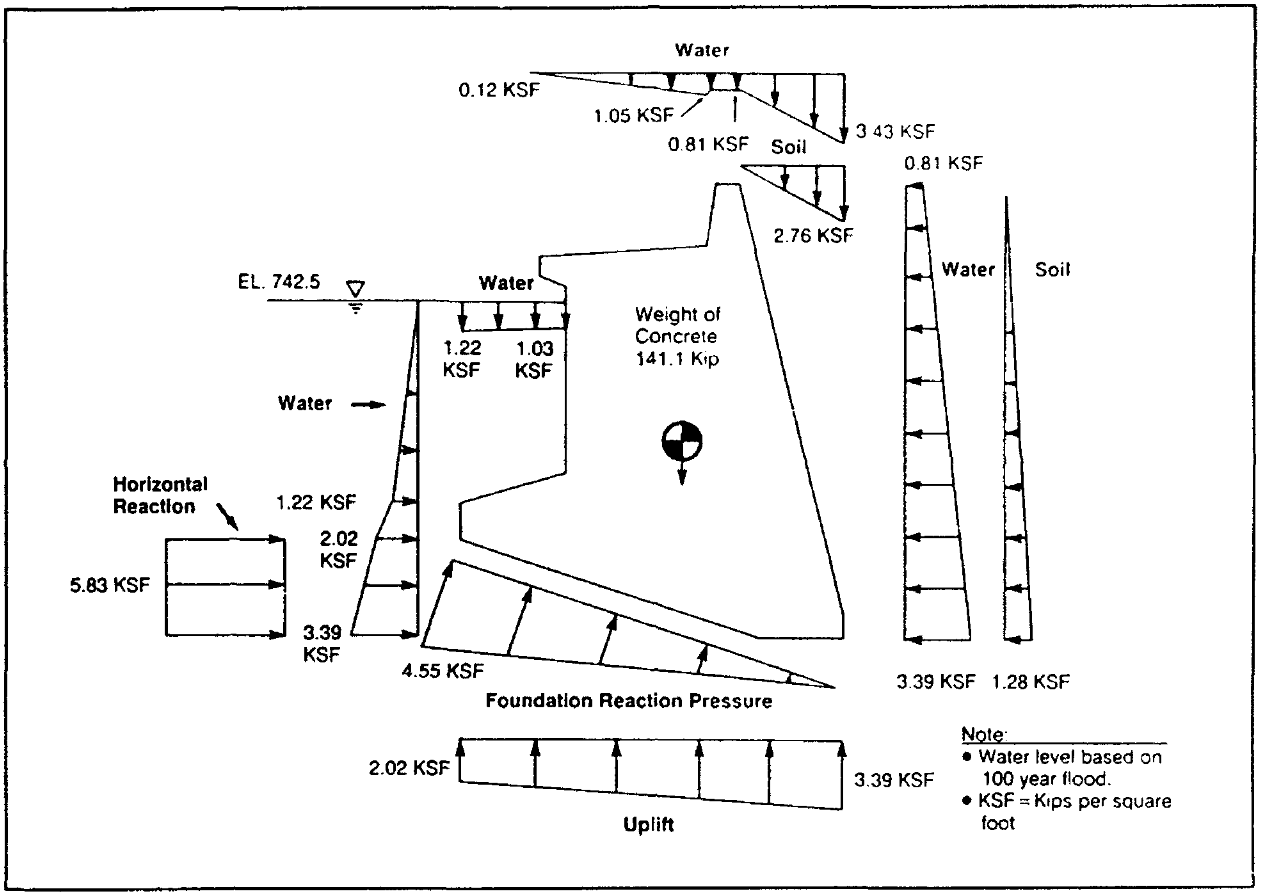

Figure 5. Fish barrier overturning stability extreme loading condition

Interim fish collection

To meet the fish collection mitigation requirements, an interim fish trap had to be in place by February 1989. In anticipation of the requirements, the contract drawings showed a yet-to-be-designed interim fish trap installed on the downstream end of the fish ladder. Unlike the earlier interim facilities, this interim trap was designed, constructed, and installed on schedule and successfully trapped salmon and steelhead.

Changes

Designing a facility where everything works on the first try was considered to be a highly desirable goal, a goal that was not attained on this job, however. Over 30 contract modifications were necessary to correct problems caused by the site changes and unforeseen requirements. This summer a follow-on contract should satisfy all remaining requirements for running the FCF.

The follow-on contract will add flow vanes to reduce sediment buildup in the channels; stop logs for future operations with the degraded channel; a new and improved vacuum system for removing sediment from the collection pool; and lifting beams for placing and removing the fish barrier notch stop logs.

Costs

Originally, we estimated building the facility would cost $1,000,000. We based cost on a facility founded on rock and similar to a $700,000 facility proposed at Elk Creek in southern Oregon. The change from rock to sediment foundations, increasing flow capabilities, unforeseen sediment transport, a high level of flood protection during diversion, dewatering, and future channel degradation led

to a design that required a construction contract of $6,700,000. An additional $1,040,000 in modifications brought the original contract to $7,750,000. With follow-on work, the total construction cost will be $8,000,000.

Lessons Learned

When designing a dam, the costs of the dam's impacts must be considered. When this project was authorized, the impact of blocking the stream was recognized and Congress authorized the decision to mitigate the blockage by building the FCF. Basic requirements of the FCF were not initially established nor were the engineering and construction efforts adequately identified. The lesson learned is that, during the planning phase, adequate time must be spent establishing the requirements of a design. Resource, budget, and schedule must be provided early in the project development to support the commitment to quality.

Key requirements need to be established and quality engineering conducted before an accurate cost estimate can be made. In this instance, requirements and conditions related to site location, foundation, streamflows, sediment, and fish collection were not recognized in the early stages and significant cost growths resulted. Pressure to provide an estimate based on limited information should be resisted until the basic site specific requirements are known.

Emphasis should be placed on site investigations. Considerable time, resources, and expense can be saved if the final site is selected before detailed design begins. In the case of the permanent FCF, schedule slips led to the need of an interim facility. The work on the interim facility meant further delays because the staff was busy designing the interim facility instead of designing the final facility.

The agencies should be consulted early and often during the project. Find out their requirements. Obtain an early committal to schedules. This committal also means timely reviews by all concerned and adequate time to do the reviews. Try to resolve issues at the working level. If the issues cannot be resolved, bring the issues to the attention of the "decision makers." Maintain good communication lines. It was a mistake to allow the agencies to review only the completed designs. If the completed designs do not meet the agencies' requirements, unplanned effort must be expended to change the design. Frustration mounts and adversarial roles develop. Establishing requirements, communicating with the customer, and resolving issues in the beginning of the design process should turn our tasks into quality projects that create enjoyable work.

Passaic River Flood Protection Project

by
Douglas J. Pendrell[1]

Abstract

The authorized project is located in the Passaic River Basin in northern New Jersey. It will consist primarily of a tunnel system to divert floodwaters beneath the heavily developed areas of northeastern New Jersey, thereby providing flood protection for these areas. The major elements of the project are two tunnels, with the key element being the 20.1-mile-long, 40-ft-diameter main tunnel, and the other a 1.2-mile-long, 22-ft-diameter spur tunnel. Together they will divert floodwaters from within the basin to an outlet in Newark Bay. In addition to the tunnels, there are many other structural features associated with this project including the inlet and outlet structures, approximately 23 miles of levees and floodwalls with 11 interior drainage pump stations, and several gated river diversion structures. The estimated construction cost is $1,790,000,000. This paper provides a description of the project, and the preliminary design and construction concepts.

Introduction

The Passaic River Basin occupies a 935-square-mile area in northeast New Jersey and southeast New York (Figure 1). Parts of eight New Jersey and two New York counties containing 132 municipalities lie within the basin. The basin has historically been subjected to serious flooding. It is now one of the more densely developed parts of the country and sustains about $100 million in damages each year on average. If the 1903 flood (the flood of record) were to occur in this area under current development conditions, an estimated $2.1 billion (in October 1990 dollars) in flood damages would result. A storm on 5 April 1984 resulted in the loss of three lives, the evacuation of more than 6,000 residents, and an estimated $390 million in damages. Over the years, the US Army Corps of Engineers has developed several flood reduction plans, the first dating back to 1939. None of these plans were implemented. The plan now under design was authorized for construction under the Water Resources Development Act of 1990. The major elements of the project are two tunnels, a 20.1-mile-long, 40-ft-diameter main tunnel and a 1.2-mile-long, 22-ft-diameter spur tunnel. The main tunnel will carry floodwaters from an inlet at the upper Pompton River down to an outlet in Newark Bay. The spur tunnel will convey Central Basin floodwaters from an inlet just downstream of Two Bridges on the Passaic River to an underground connection with the main tunnel. Five gated structures will regulate flow in the channels and into the tunnels. About 5.9 miles of channel modifications, 17.6 miles of levees, and 6.2 miles of floodwalls will increase flood protection. This paper focuses on the tunnel and the gated structures. The project is in an early phase of Preconstruction Engineering and Design and the designs

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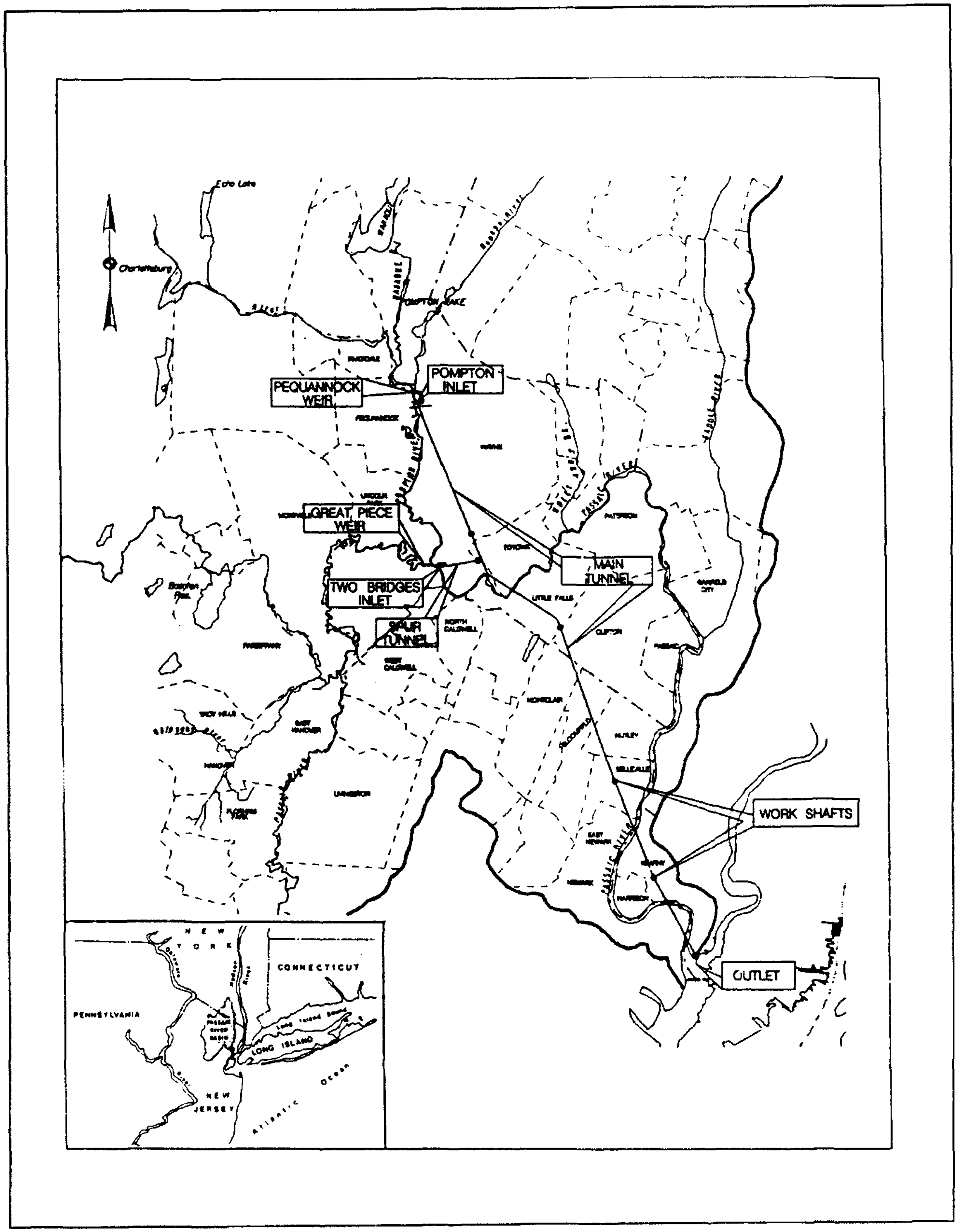

Figure 1. Passaic River flood protection project

presented in this paper are preliminary. They will likely be modified as investigations and analyses are completed.

Geology

Physiography and topography

The Passaic River Basin Tunnel Project is located completely within the Appalachian Physiographic Province. The Appalachian province is divided into six lesser provinces, two of which, the Highlands and Piedmont, contain the watershed. The Highlands province, with its altitudes ranging from 600 to 1,500 ft, is highly dissected with high relief. The tunnel route is located entirely within the Piedmont province, which is topographically low and smooth in relief. The topography is formed by glacial material which covers the region and results in gently rolling and undulating plains for the most part. Two hundred to three hundred feet above the plains are the Watchung Mountains formed by resistant basalt flows. These mountains are three elongate ridges with crest elevations ranging from 450 to 870 ft.

Stratigraphy

In New Jersey, the Piedmont province is known geologically as the Newark Basin. Sandstones, shales, limy shales, and conglomerates were deposited in the Newark Basin during Triassic and Jurassic times. Interspersed with these sedimentary deposits are three separate igneous basalt beds which were laid down as sheet lava flows. The basalt and sedimentary units alternate within the Passaic River Basin with the oldest formation in the east and progressively younger formations toward the northwest.

Structural geology

The prominent structural feature relative to the tunnel design and construction is a series of faults in the vicinity of the Watchung Mountains. The faults generally strike in a north-south direction and dip toward the east at relatively high angles. Five faults have been observed or inferred thus far. Future explorations will attempt to define the number and extent of fault zones the tunnel will have to penetrate.

Glacial geology

Continental glaciation occurred in the Passaic River Basin during the Kansan, Illinoian, and Wisconsin stages. The present landscape of the basin was created by the last stage of the Wisconsin glacier. Surficial deposits consist of glaciofluvial and lacustrine soils ranging from gravel to clay. Available drill hole data indicate that there are buried valleys along the tunnel route ranging in depth from 30 to nearly 300 ft below the present land surface. It is critical that the buried valleys along the tunnel route be located prior to construction.

Tunnel geology

The alignment of the Passaic River Basin Tunnel cuts across the strike of the Newark Basin (Figure 2). The tunnel outlet, in the vicinity of Kearny Point on Newark Bay, is the lowest point in the geologic sequence. Proceeding upsection from the outlet, the tunnel will traverse the Passaic Formation, the Orange Mountain Basalt, the Feltville Formation, the Preakness Mountain Basalt, the Towaco Formation, the Hook Mountain Basalt, and finally the Boonton Formation. Overall, the main tunnel will be excavated through approximately 93,000 linear feet of sedimentary rock consisting primarily of fine sandstone, siltstone, and shale and about 15,000 linear feet of basalt. It is not yet known how much of the tunnel will be excavated through fault zones. It is presently estimated at 250 to 500 ft. The tunnel runs nearly perpendicular to the faults, which will minimize the length of excavation through this less suitable material. The spur tunnel will encounter the Preakness Mountain basalt and the Towaco Formation.

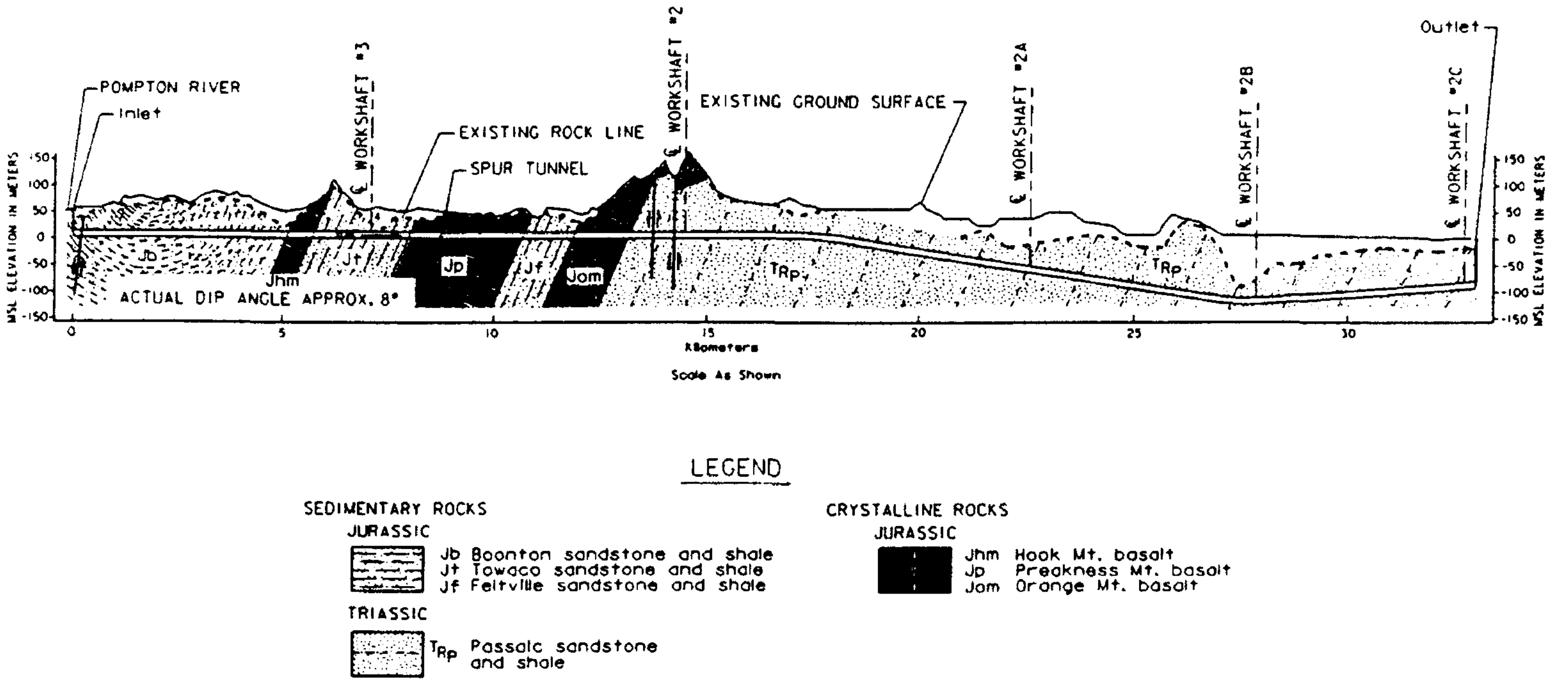

Figure 2. Geologic profile along the Passaic River Basin Tunnel alignment

Project Description

The authorized project consists of four structurally distinct but integrated key components: the main tunnel, a spur tunnel, levees and floodwalls, and channel modifications (Figure 1). In addition, there are the Great Piece Meadows Weir (shown in Figure 1 as Great Piece Weir) and the Pequannock Weir, which will control upstream water levels. As noted earlier, the levees and floodwalls and channel modifications will not be addressed in this paper.

Tunnels

The function of the tunnels is to divert flood flows from the upper Pompton River and the central Passaic River to an outlet in Newark Bay. The main tunnel will be about 20.1 miles long. The inside diameter will be about 40 ft, with an excavated diameter of nearly 44 ft. The spur tunnel will be about 1.2 miles long, with an inside diameter of 22 ft and an excavated diameter of nearly 26 ft. The tunnel depth will vary from about 125 to 450 ft below the ground surface. About half of the tunnel invert will be just above sea level so it will remain drained when not in operation. However, due to the existence of buried bedrock valleys near the lower end, the invert will drop to approximately 450 ft below sea level to maintain an adequate rock cover over the tunnel crown (Figure 2).

Intake structures

The inlet to the main tunnel (Pompton Inlet) consists of a flow restrictor, diversion spillway, and morning glory inlet (Figure 3).

The function of the flow restrictor is to regulate flows past the tunnel inlet. The structure is designed with four 16.5- by 28-ft openings and one 16.5- by 32-ft center opening, all of which are culverts 30 ft in length. The openings are controlled by roller gates which are withdrawn into the abutments of the flow restrictor. The restrictor is located on the river downstream of the diversion structure. The top of the structure serves as an emergency spillway when the design flood is exceeded. Embankments extend outward from the structure almost 720 ft on the east bank and 750 ft on the west bank and average 4 to 5 ft in height.

The purpose of the diversion spillway is to regulate the flow to the tunnel during flood events and to maintain existing flows in the river during normal conditions. The 400-ft-long ogee-shaped spillway is designed with five 5.5- by 75-ft bascule gates. The crest with the gates down is 5 ft above the channel bottom and 8.7 ft above the access basin bottom. With the gates up, the structure will allow bypass of the 1-year flow with no diversion into the tunnel. Flows larger than the 1-year flood would be diverted into the tunnel.

The morning glory inlet is located behind the diversion spillway in a large semicircular basin about 400 ft in diameter and 8 ft deep relative to the morning glory crest. The access basin is designed to provide sufficient space for the flow to distribute around the morning glory spillway and to reduce the velocity head approaching the spillway. The crest of the spillway has a radius of 42 ft and a crest length of 264 ft. The morning glory spillway transition to the 125-ft-deep tunnel intake shaft is designed as a streamlined shape to provide a smooth, gradual change to minimize head losses and to avoid zones where cavitation pressures can develop. Guide piers on top of the morning glory spillways are designed for antivortex action and to maintain converging flow into the drop inlet.

The inlet to the spur tunnel (Two Bridges Inlet) is similar to the Pompton inlet but does not include a flow restrictor. It consists of the diversion spillway and the morning glory inlet (Figure 4).

This structure is similar in appearance and function to the diversion spillway at the Pompton Inlet. There are four 11- by 100-ft bascule gates designed for the 420-ft-long Two Bridges diversion spillway structure. The crest is 1.7 ft higher than the improved channel bottom and 11.6 ft higher than the access basin bottom. The top of the gates are 4 ft

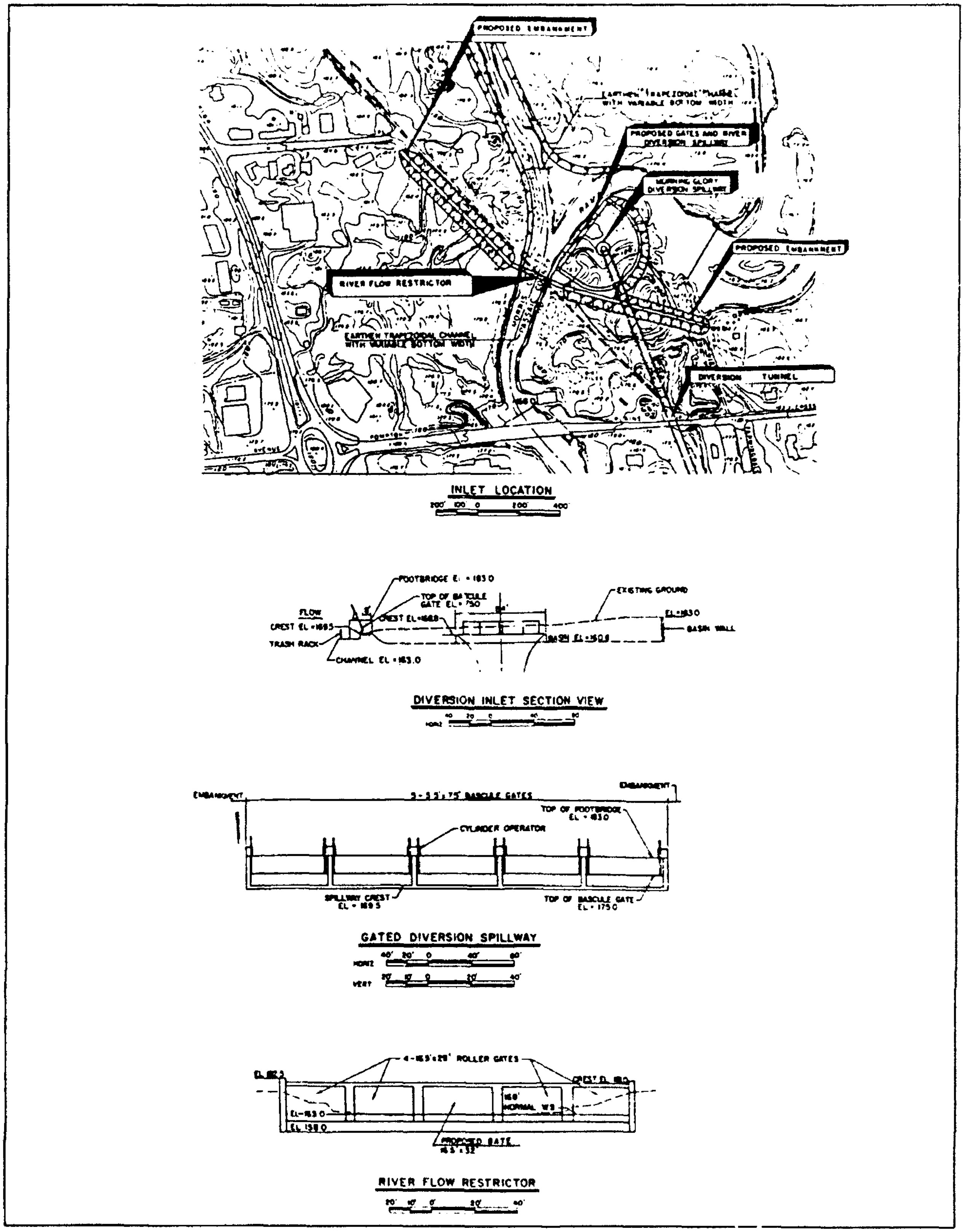

Figure 3. Pompton Inlet

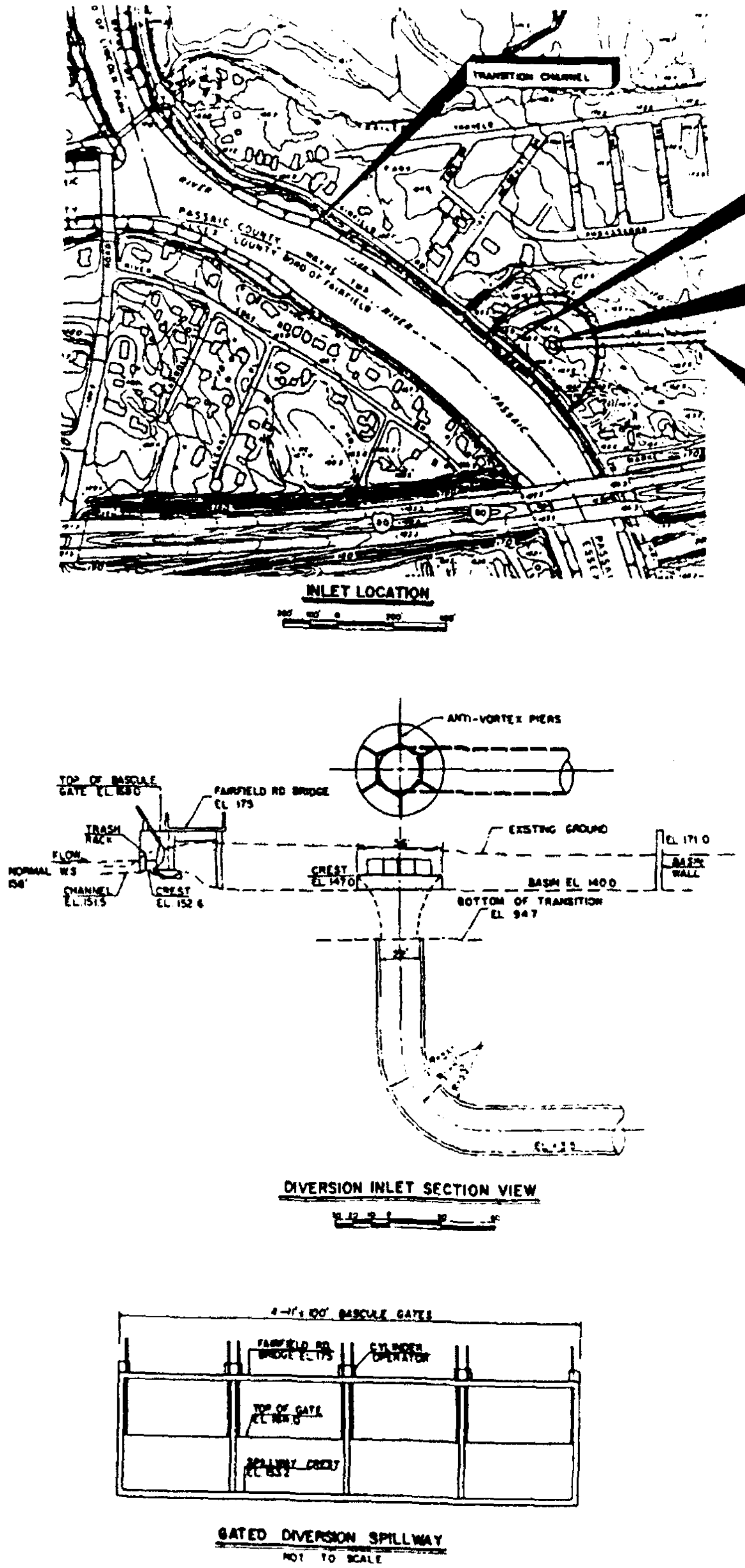

Figure 4. Two Bridges Inlet

higher than the existing 1-year flood stage. The crest elevation was determined to be the stage of the 100-year bypass flow when the gates must be closed (plus 2 ft of freeboard) to maintain the priority of the Pompton Inlet diversion.

The morning glory spillway is designed in accordance with the same hydraulic criteria as the Pompton Inlet structure. Specifically, this structure is situated in a 280-ft-diameter semicircular access basin which widens at its mouth to 412 ft. The 28-ft-radius spillway has a crest length of 182 ft, and is 7 ft above the floor of the access basin. The depth of the intake shaft is about 130 ft.

Pequannock River weir

This structure is included to maintain normal river flow stages under normal conditions on the Pequannock and Ramapo Rivers upstream of the Pompton Inlet. It will consist of two 85-ft-wide, 15-ft-high bascule gate sections which would be raised under normal conditions and lowered during a flood event.

Great Piece Meadows weir

This weir is incorporated into the design for purposes of channel stability and to assure appropriate inundation of wetlands within the 1-year floodplain upstream of the weir. The weir would consist of one 100-ft-wide, 14.7-ft-high bascule gate. It would be founded on the existing channel bottom 3,630 ft upstream of Two Bridges Inlet. A 600-ft-long embankment would tie the weir into the left bank and a 150-ft-long embankment would tie into the right bank.

Outlet structure

A conceptual design for the outlet structure in Newark Bay has not yet been developed. This is due to the fact that the project authorization in November 1990 revised the alignment and outlet location so that the preliminary design for the preauthorized outlet is obsolete. It will, however, require a 450-ft upshaft and a structure that will minimize impacts to Newark Bay.

Work shafts

In addition to the inlet and outlet shafts, four work shafts and two access shafts are planned for access and egress. Five of these are located along the main tunnel and one is located on the spur tunnel near its junction with the main tunnel. The shafts are spaced to limit tunnel runs from the shafts to 4 miles or less although one segment will be slightly longer. Shaft diameters will vary between 10 and 35 ft depending on their function.

Design Considerations

Rock properties

During the early feasibility studies, unconfined compressive tests have been conducted on the rock materials that are present along the tunnel alignment. The test data indicate a range from 5,450 psi to 34,220 psi with an average 14,010 psi for the four sedimentary formations. The three basalt formations range from 7,720 psi to 48,580 psi with an average 23,370 psi. Overall, the rock quality at the tunnel elevation can be considered good to excellent for tunnelling. It is relatively massive and only moderately jointed with many of the joints healed. However, it should be noted that considerably more investigations and testing are planned to confirm these assumptions.

Fault zones

Early investigations have provided information on the condition of the faults through which the tunnel will be constructed. The actual faults are relatively narrow, ranging from about 6 in. to 4 ft. They contain varying degrees of clay gouge and breccia. Extending outward from the faults are zones that exhibit a relatively high degree of jointing and brecciation. The joints and breccia interstices in these zones are healed with calcite or gypsum to the extent that the rock cored relatively solid and unbroken. These zones appear to extend from as little as 5 ft to as much as 50 ft to either side of the fault. It can be assumed that poorer quality rock will be encountered for some distance to either side of the faults.

Evaluation of these fault zones is a primary objective of future investigations.

Tunnel support and lining

The evaluation of the tunnel stability and the design of the temporary support requirement will follow a sequential development. The first phase of the analysis will involve the use of two rock mass classification systems (Bieniawski 1979): the Q-SYSTEM, developed by the Norwegian Geotechnical Institute, and the RMR-SYSTEM, developed by Bieniawski. For each of these systems it will be necessary to divide the tunnel length into sections based on variations in the rock mass quality as predicted from the exploration results. For each of these sections, the appropriate coefficients for various rock mass properties such as strength, weathering, water inflow, etc., are selected. These numbers are used in a simple equation to predict the amount and type of support required to stabilize the opening. These systems have been developed using large empirical data bases and are normally a very conservative design approach.

Confirmation of the rock mass classification designs will be provided in two phases which will include computer analysis of the tunnels. In the first phase, a boundary element stress analysis will be conducted to approximate the stress field surrounding the tunnels and how the rock mass will respond. Input into this analysis will include the rock mass strengths and the in situ stress field. The location and length of the support, i.e., rock bolts, shotcrete, or steel ribs, can then be evaluated. Results of this analysis are used to spot-check the Rock Mass Classification System results.

The second phase permits the modeling of nonlinear, nonelastic behavior which is not possible with the boundary element analysis. Several variations of this type of analysis are available. During this phase the interaction of the support system with the rock mass, discontinuities, and stress field will be predicted. Output from this program includes a displacement diagram which shows the predicted magnitude and direction of movement within the rock mass as a result of the tunnelling.

With the present level of exploration it appears that at least 99 percent of the main tunnel and 100 percent of the spur tunnel have rock quality designations (RQD's) greater than 85 percent and can be supported initially by rock bolts and shotcrete. All fault zones will most likely need steel ribs with blocking and lagging to support the brecciated and disturbed rock. During the final exploration the rock along the tunnel alignment will be divided into classification relating to its support requirements.

Lining Design

Although the final lining design is yet to be made and precast segmental lining will be considered, it is most likely that a cast-in-place lining will be used on this project. That being the case, unreinforced concrete lining will be used in those sections of the tunnel that are in competent rock. This will represent by far the majority of the tunnel length. The less competent areas such as the fault zones may require steel-reinforced concrete lining. Preliminary analysis indicates a 12-in.-thick liner will satisfy structural strength requirements. However, an additional 9 in. are necessary where steel ribs are installed to ensure adequate imbedment. Therefore, to maintain a uniform cross section necessary for tunnel boring machine (TBM) excavation, the preliminary lining design is 21 in.

Shafts

The work shafts, inlets, and outlet will be excavated through soil overburden and rock to the appropriate tunnel elevation. Preliminary data indicate the amount of overburden that will be excavated for each shaft varies from as little as approximately 25 ft at work shaft 2 to as much as approximately 300 ft at shaft 2B. Investigations and design for the shafts have not yet been accomplished. Groundwater inflow and sidewall stability are the primary concerns that will be addressed during explorations and design.

Tunnel Construction Methods

Although conventional excavation using drilling and blasting techniques is clearly feasible and would offer flexibility in case adverse geologic conditions were encountered, it appears that a TBM is clearly the most economical method to approach the Passaic River Flood Protection Project. Even though a TBM of approximately 44 ft in diameter is unprecedented at this time, it is believed to be well within the technology of the manufacturers to produce such a machine. The geologic conditions present along the alignment are believed to be suitable for this alternative also. All the well-known positive reasons for TBM excavation such as increased production, less temporary support, and less concrete lining as the result of reduced overbreak, are present on this project. With such a large project it will be necessary to have several contracts with two or three TBM's working, some concurrently. The spur tunnel will be awarded as a separate contract and can be constructed either by drill and blast or TBM methods.

The TBM's proposed for use on the Passaic Tunnel are larger than any built to date. In addition to the unprecedented size, the machines will be equipped with larger bearing pads for support and grip of the softer sedimentary rocks. They will have a probe hole capability for drilling ahead of the face and drilling rigs installed on either side of the main beam for rock bolt installation. There will be a rib-erector system included as well as the capability to install support and rock reinforcement immediately behind the cutter head. The cutter heads will be equipped with back-mounted, recessed cutters.

Conclusion

The Passaic River Flood Protection Project is in a very early phase of design. The remarkable size and length offer designers and future contractors a unique challenge. It is an understatement to say that such a project will require a great deal of planning and design to accomplish. However, with attention given to careful design and construction, a project of great benefit can result for the people of the Passaic River Basin.

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Rio Puerto Nuevo Flood Control Project San Juan, Puerto Rico

by
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Abstract

The Rio Puerto Nuevo basin is within the San Juan Metropolitan Area. The study area includes the entire basin of the Rio Puerto Nuevo and its tributary streams Margarita, Josefina, Dona Ana, Buena Vista, and Guaracanal. The proposed channel improvements would consist of 11.2 miles of channel improvements, two debris basins, a road relocation, the replacement of 17 bridges, the modification of 8 bridges, and the construction of 5 new bridges.

The channel improvements would provide 100-year flood protection. The majority of the project would consist of high-velocity, supercritical, concrete channels. Physical models of the regions of supercritical flow were constructed and analyzed by the US Army Engineer Waterways Experiment Station (WES) to verify the mathematical model.

Design challenges for the proposed channel wall systems involve the variable foundation and water conditions. Types of construction include king pile retaining walls, sheet-pile walls, reinforced concrete T-walls and U-shaped channels. Additional design objectives include minimal disruption of traffic, construction within highly congested areas, and the incorporation of recreational features and a mangrove mitigation plan into the project design.

Introduction

The Rio Puerto Nuevo survey investigation was initiated in 1978 at the request of the Commonwealth of Puerto Rico. It was conducted under the authority of Section 204 of the Flood Control Act of 1970 (PL 91-611). The Rio Puerto Nuevo, Puerto Rico, General Design Memorandum (GDM) was authorized as part of the Water Resources Development Act of 1986, Public Law 99-662, November 17, 1986. The overall purpose of this study was to investigate the water and related land resource problems along the Rio Puerto Nuevo and its principal tributary streams and to develop the most desirable plan for solving these problems.

The Rio Puerto Nuevo basin is located within the San Juan Metropolitan Area, which has a population of over one million. The study area includes the entire basin of the Rio Puerto Nuevo and its tributary streams Quebrada Margarita, Quebrada Josefina, Quebrada Dona Ana, Quebrada Buena Vista, and Quebrada Guaracanal.

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The basin covers an area of approximately 62.8 square kilometers (25 square miles). It drains into the San Juan Harbor. Over 75 percent of the area is developed, and by the year 2000, it is expected that all undeveloped lands would be urbanized.

Intense development in the drainage basin has altered the natural drainage patterns, significantly increasing the runoff water and restricting the flows in the flood plain. The existing channels and numerous bridges are undersized and in poor condition. Development extends to the banks of the channels and hinders flood flows. Extensive damage is caused by any flood that exceeds the 2-year storm. Improvements are needed for the channels and bridges located within the study area.

The total first cost of construction for the proposed project improvements is $303.2 million. Based on the current interest rate of 8-3/4 percent and a 50-year project life, project implementation would result in $73.8 million in annual benefits. The overall benefit-to-cost ratio is 2.4 to 1.

Description of Proposed Improvements

The channel improvements along Rio Puerto Nuevo involve deepening, widening, and straightening 6.5 miles of the main channel from its outlet in the San Juan Harbor to Winston Churchill Avenue and the construction of a debris basin. The width of the channel varies from 400 ft in the lower reach to 60 ft in the upper reach. The Puerto Nuevo Channel is divided into five reaches: from Sta. 0+90 to Sta. 88+33.2 - the Lower Bulkheaded Channel (Reach 1); from Sta. 88+33.2 to Sta. 147+40 - the Lower Subcritical Channel (Reach 2); from Sta. 147+40 to Sta. 184+48.51 - the Composite Section Supercritical Channel (Reach 3); from Sta. 184+48.51 to Sta. 276+50 - the Upper Supercritical Channel (Reach 4); and from Sta. 276+00 to Sta. 341+92.13 - the Upper Subcritical Channel (Reach 5).

The reach from the San Juan Harbor to the confluence with Margarita would be widened and deepened from the existing 180-ft-wide and 10-ft-deep channel to a 400-ft-wide and 25-ft-deep channel. This reach is approximately 1.65 miles long and experiences tidal action. Design velocities range from 6 to 9.5 fps.

Several factors influenced the alignment of this reach. At the Harbor, an existing mangrove/mudflat system along the west bank was a valuable wildlife habitat. The proposed improvements would maintain the existing west bank and widen 400 ft eastward. The existing mangrove/mudflat system would be designated a National Reserve and managed by the Commonwealth.

A king pile wall system would be constructed from Sta. 3+80 to Sta. 57+40. The channel walls would provide protection to the mangroves from the wake of boats expected from the Agua-Guagua project. The top of the channel would be at el 3,[1] however some areas would have reduced wall heights just below the water surface to allow for tidal flushing necessary for the mangroves to thrive. The existing bulkhead wall along the west bank would not require removal and would be incorporated into the channel improvements.

The new king pile wall system would be similar to that constructed under the Martin Pena contract. Material behind the wall is limited to el 0. Along the east bank, a 60-ft-wide berm at el 0 would be constructed to provide a area for mangrove replanting.

The San Juan Municipal Landfill extends along the west bank of Rio Puerto Nuevo, from Kennedy Avenue to the confluence with Margarita. From Sta. 57+40 to Sta. 74+32.5, the landfill encroaches into the existing channel right-of-way. The original alignment would have required extensive excavation

[1] All elevations (el) cited herein are in feet referred to National Geodetic Vertical Datum (NGVD) of 1929.

into the landfill. In addition, the channel is bounded by the De Diego Expressway on the east bank.

Various alignment alternatives were considered. A localized reduction in the channel width and then widening downstream to compensate for the resulting backwater effect was investigated. This alternative was the most costly due to the increased real estate, excavation, and erosion control requirements.

The selected alternative involved the relocation of approximately 1,500 ft of De Diego Expressway. This alignment alternative would avoid excavation in the landfill and only involve two lanes of traffic. These two lanes would be relocated approximately 100 ft east of the existing road, and construction could be accomplished with minimal disruption of traffic. Cost comparisons of the alignment alternatives determined that the De Diego Expressway relocation alternative was the most cost effective.

A steel sheet-pile wall with a concrete facing is proposed for Sta. 57+40 to Sta. 74+32.5. Geotechical investigations concluded that driving concrete sheet piles in this reach was not feasible. The concrete facing would provide a relatively smooth surface which is desirable hydraulically as well as provide corrosion protection. Riprap protection would be provided at the toe of the sheet pile and extend along the berm and side slopes. Material behind the wall is limited to el 3. From Sta. 74+32.5 to Sta. 88+33.2 a trapezoidal earth channel is proposed. Along the west side, a 60-ft-wide berm at el 0 would be constructed within the channel to provide a mangrove replanting area.

The proposed channel improvements for Reach 2 consist of widening and deepening the existing channel with a channel width ranging from 180 to 150 ft and an invert elevation ranging from -16 to -12.7 ft. The design velocities range from 9 to 15 fps. While this reach is within the subcritical regime, concrete channels would be necessary to provide the required protection and smoothness. This reach is within the zone of tidal influence and it is not considered feasible to dewater to construct a conventional concrete channel. A relieving platform wall constructed of steel sheet pile with a concrete facing and a tremie concrete bottom is proposed.

The proposed channel improvements for Reach 3 consist of widening and deepening the existing channel with a channel width ranging from 150 to 102 ft and an invert elevation ranging from -12.7 to 11.5 ft. The design velocities range from 12.4 to 38.9 fps. A stilling area with baffle blocks would be constructed from Sta. 149+60.18 to Sta. 147+41.18. A lowered invert center section would be constructed from Sta. 184+48.51 to Sta. 155+78.52. The lowered invert center section would be 29 ft wide with an invert 10 ft lower than the proposed channel invert.

There are four major existing bridges in this reach; Las Americas Expressway, Pinero Avenue, and the NE and SE Access Ramp bridges. The composite section was designed to maximize the conveyance through the bridges. Side slopes were established to parallel existing abutment and pier piling.

It was desirable to maintain the cross section dictated by the bridges to extend through the entire reach in order to minimize head losses. The construction of the channel walls would be accomplished by using a buttressed wall system. Stabilization of the bridge side slopes would be accomplished by soil nailing and shotcrete. A cast-in-place concrete channel wall would be placed against the soil nailed face.

The downstream portion of this reach, at the stilling area, is within the zone of tidal influence. Construction methods utilized in Reach 2, which would not require dewatering, were not considered feasible in this area due to hydraulic requirements. Conventional reinforced concrete T-walls and bottom slab are proposed for the stilling area and the remainder of Reach 3, with buttresses where necessary.

The proposed channel improvements for Reach 4 consist of widening and straightening the existing channel with a channel width ranging from 107 to 60 ft. The proposed channel invert is 5 to 10 ft higher than the existing channel bottom with an proposed channel invert ranging from 11.5 to [illegible].4 ft. The design velocities range from 24.1 to 39.9 fps. Conventional reinforced concrete T-walls and bottom slab are proposed.

The proposed channel improvements for Reach 5 consist of widening, deepening, and straightening the existing channel with a channel width ranging from 60 to 75 ft and an invert elevation ranging from 53.4 to 60.5 ft. The design velocities range from 12.7 to 24.1 fps. Conventional reinforced concrete T-walls and bottom slab are proposed. Included in the channel improvements is the construction of a debris basin and a side over-flow spillway. The spillway would be of similar construction as the channel improvements.

The proposed channel improvements for Margarita would consist of a earthen trapezoidal channel with 1V on 6H side slopes from Sta. 0+00 to Sta. 39+00 and 1V on 8H side slopes from Sta. 40+00 to Sta. 54+10. The channel bottom width is 30 ft wide and the invert elevation ranges from -16.0 to -14.9 ft. The design velocities range from 1.9 to 4.4 fps. A small dike would be constructed along the north bank to provide the required protection elevation. A concrete wall would be constructed on the south side of the tributary at the confluence of Margarita and Rio Puerto Nuevo to provide a transition from the rectangular channel section and the trapezoidal section.

The proposed channel would be constructed north of the existing Margarita. The existing Margarita parallels De Diego Expressway. By constructing the channel improvements north of the existing tributary, the existing drainage systems could remain in place without modification and no impact on the Expressway would be realized. An inlet for the existing Margarita would be provided at its confluence with the Puerto Nuevo Channel.

Within the proposed channel cross section, a 60-ft-wide berm at el 0.0 ft would be constructed to provide a mangrove replanting area.

From Sta. 54+10 to Sta. 89+60 the proposed channel improvements would consist of a rectangular, concrete u-shaped channel, with a 30-ft bottom width and a channel invert ranging from elevation -17.87 to 12.60. Design velocities range from 38.4 to 20.5 fps. From Sta. 54+75 to Sta. 54+10 a stilling area would be constructed. Riprap protection would be provided at the downstream end of the stilling area. At Sta. 89+60 the proposed channel improvements would tie into an existing concrete trapezoidal channel.

The proposed channel improvements for Josefina consist of deepening the existing channel 20 to 5 ft deeper than the existing invert. From Sta. 0+00 to 34+36.72 the channel width would be 45 ft and the invert ranges from -13.5 to -1.4 ft. Design velocities range from 8.3 to 24.4 fps. In this reach the proposed method of construction would be conventional reinforced concrete T-walls and bottom slab.

From Sta. 34+36.72 to Sta. 77+28.84 the proposed channel width varies from 22 to 17 ft and invert ranges from el -1.44 to 27.40. The design velocities range from 17.1 to 26.4 fps. From Sta. 34+36.72 to Sta. 36+78.52 a reinforced concrete u-shaped channel is proposed. From Sta. 36+78.52 to Sta. 77+28.84 the proposed channel would be constructed within the existing channel. Steel sheet-pile channel walls with concrete facing and a reinforced concrete slab are proposed. Headwalls are proposed at the upstream end of the channel improvements which tie into high ground to provide an inlet to the design channel.

The proposed channel improvements for Dona Ana consist of deepening the existing channel 20 to 5 ft deeper than the existing channel invert. The proposed bottom width is 22 ft and the invert ranges from el -1.4 to 36.5 ft. The design velocities range from 18.1 to 26.9 fps. From Sta. 0+00 to Sta. 5+30.83 a

reinforced concrete u-shaped channel is proposed. From Sta. 5+30.83 to Sta. 32+80 the proposed channel would be constructed within the existing channel walls. Steel sheet-pile channel walls with concrete facing and a reinforced concrete slab are proposed.

A new channel is proposed in lieu of improving the existing Buena Vista which traverses through a heavily developed residential area. Construction of the diversion channel would eliminate the excessive loss of homes as it would be constructed in a relatively undeveloped tract. The proposed channel width would range from 16 to 36 ft with an invert elevation ranging from 22.3 to 31.18 ft. The design velocities would range from 23.0 to 6.1 fps. A reinforced concrete u-shaped channel is proposed. Headwalls are proposed at the upstream end of the channel which tie into high ground to provide an inlet to the design channel. The proposed channel improvements for Guaracanal would consist of straightening the existing tributary with a channel width ranging from 26 to 50 ft and an invert elevation ranging from 41.5 to 46.3 ft, NGVD. The design velocity would range from 29.6 to 8.4 fps. The proposed channel would be a reinforced concrete u-shaped channel from Sta. 0+00 to Sta. 5+23.06 and a reinforced concrete T-wall and bottom slab from Sta. 5+23.06 to Sta. 8+19. The channel improvements include the construction of a debris basin and a side overflow spillway. The construction of the spillway would be reinforced concrete T-walls and bottom slab. The debris basin would require the construction of a dike and floodwalls where the necessary easements for dike construction are not available.

The proposed channel improvements involve 30 bridges. Eight of these bridges are existing bridges which would not require replacement but would require abutment and pier protection due to widening and deepening the channel. Costs for the protection of these bridges is included in the channel improvement costs. Seventeen bridges would require replacement, fifteen highway bridges and two pedestrian bridges. Five new bridges are required, three highway bridges and two pedestrian bridges. Costs for these bridges would be included in the relocation costs, which would be the responsibility of the Commonwealth of Puerto Rico.

Structural Design

The recommended plan of improvements includes several proposed channel sections. The selection of channel section is dependent upon the various soil and water conditions encountered as well as external construction limitations.

The king pile channel wall is proposed for Rio Puerto Nuevo from Sta. 3+80 to Sta. 57+40. The walls would consist of 24-in., longitudinally slotted, prestressed concrete vertical piles, driven at 8-ft centers with 18-in. square, prestressed concrete, battered piles driven behind the wall as a tieback. Prestressed concrete panels, 6 in. thick, would be inserted in the slots and span the vertical piles to retain the soil behind the wall. The average length of the vertical piles would be 50 ft.

The sheet-pile channel wall is proposed for Rio Puerto Nuevo from Sta. 57+40 to Sta. 74+32.50. The walls would be constructed of steel sheet pile with a 6-in.-thick concrete facing. Prestressed concrete battered piles, 18 inches square, would be driven and placed at 8-ft centers as a tieback. The average length of the piles would be 40 ft.

The relieving platform channel wall is proposed for Rio Puerto Nuevo from Sta. 88+33.2 to Sta. 147+40. The walls would consist of steel sheet piling with a 6-in. concrete facing. A reinforced concrete platform would be supported partly on the sheet piling and on prestressed concrete battered piles. A 4-ft-thick slab of tremie concrete wall be placed along the channel bottom.

This type of construction was selected to reduce the lateral pressure acting on the steel sheet piling. The surcharge and a portion of the weight of the fill are carried as vertical load to a deeper level where they do not have influence on the steel sheet piling. This allows

for higher walls to be built and heavier loads to be supported within the strength limitations of the sections of steel sheet piling commercially available.

Inverted T-channel walls are proposed for Rio Puerto Nuevo from Sta. 147+40 to Sta. 150+78.52, from Sta. 179+48.51 to Sta. 338+92.13, and for Josefina from Sta. 0+00 to Sta. 34+36.72. The inverted tee walls and bottom slab would be constructed of reinforced concrete. An uplift pressure relief system would be required to prevent excessive hydrostatic head build-up beneath the structure. Foundation improvements, such as excavation and replacement of poor material or a pile foundation, would be required for various reaches.

The trapezoidal channel with lowered invert center section is proposed for Rio Puerto Nuevo from Sta. 150+78.52 to Sta. 184+48.51. The sloped channel wall would be a counterfort wall, where the base slab and wall span between vertical triangular braces (counterforts). The wall, braces, and base would be reinforced concrete as well as the bottom slab and lowered invert center section. An uplift pressure relief system would be required to prevent excessive hydrostatic head build-up beneath the structure. Foundation improvements may be required.

A concrete u-frame channel is proposed for Margarita from Sta. 54+10 to Sta. 89+60, Josefina from Sta. 34+36.72 to Sta. 36+78.52, Dona Ana from Sta. 0+00 to Sta. 5+30.83, Buena Vista from Sta. 0+00 to Sta. 42+37.79, and Guaracanal from Sta. 0+00 to Sta. 8+19. The channel would be constructed of convention reinforced concrete. An uplift pressure relief system would be required to prevent excessive hydrostatic head build-up beneath the structure. Foundation improvements would be required for various reaches.

The sheet-pile wall channel is proposed for Josefina from Sta. 36+78.52 to Sta. 77+28.84 and Dona Ana from Sta. 5+30.83 to Sta. 32+80. Steel sheet piling would be driven within the existing channel walls and braced internally with wales and struts. A 6-in. concrete facing would be applied over the exposed surfaces of the steel sheet pile. The channel would be founded on a 4-in.-thick mud slab. A concrete bottom slab would be placed over the mud slab.

There are eight bridges within the project limits which would not require replacement. Construction of the required channel cross section beneath these bridges without disrupting vehicular traffic and without replacing the bridges were the foremost objectives. Treatment of the exposed foundation or providing pier and abutment protection were the primary concerns.

The proposed channel improvements under Kennedy Avenue bridge would result in a channel with a design invert of -25 ft, a channel width of 400 ft, and a design velocity of 8.5 fps. The original substructure is cast-in-place concrete piers resting on pile foundations. The deepening and widening of the existing channel would result in the exposure of portions of the piling foundation for five of the bridge pier groups.

Individual steel sheet-pile cofferdams would be constructed around each affected bridge pier group. The cofferdams would be permanent and would retain existing soils around the existing bridge foundation piling. Each cofferdam must have sufficient width to be outside of the existing battered bridge piling. Since the depth of channel and pile batter varies at each pier, each of the five cofferdams would have different widths. The width of the cofferdams varies from 14 ft to 35 ft. The width of the cofferdams would restrict the flow opening beneath the bridge and cause increased velocities at the bridge opening.

The combined width of the required cofferdams at each pier location reduces the hydraulic bridge opening by 28 percent. This restriction would cause the design channel velocity to increase from 8.5 fps to 14.4 fps. Adequate scour protection would be provided to protect against the scour potential of the increased velocity.

The overhead clearance beneath the bridge is approximately 20 ft. The restricted overhead clearance would require that the sheet piling be spliced about every 10 ft with a full depth weld and splice plate. A compact, vibratory pile driver would be required because of the restricted overhead clearance.

Tie rods would be installed between the cofferdam walls, near the top, on 6-ft centers. The top edge of the sheet-pile cofferdams would be encased in a reinforced concrete cap to strengthen the cofferdam and protect the upper portion of the steel sheet pile from the more aggressive corrosion action experienced in the tidal zone.

The required channel cross section under Las Americas Expressway bridge would be constructed by stabilizing the slope using soil nailing and then providing a cast-in-place reinforced concrete channel wall and slab. Soil nailing would be constructed by staged excavations from top down. Shotcrete would be sprayed onto the exposed face. Holes would be drilled for the soil nails, usually reinforcing bars, and the nails would be inserted and grouted. A second layer of shotcrete would be applied.

The proposed construction method was selected due to the very limited clearances available. The construction equipment required for soil nailing would be drilling rigs for reinforcement installation and guns for shotcrete application which are relatively small scale, mobile, and quiet. Soil nailing could proceed rapidly and would be applied at the earliest possible time after excavation. This minimizes the disturbance to the ground and the possibility of damage being caused to the bridge.

Additional improvements to the bridge include pier protection and bridge pier extensions. The individual column piers would be encapsulated in concrete to provide a single pier. These piers would be extended beyond the upstream face of the bridge and shaped to deter trash accumulation. Similar construction methods are proposed for the NE Access Ramp, Pinero Avenue, and the SE Access Ramp.

Corrosion Mitigation Investigations

A corrosion mitigation investigation was conducted along a 1.7-mile reach of Rio Puerto Nuevo, from the San Juan Harbor to the confluence with Margarita. This area was of greatest concern due to the tidal action, proximity to industry and the municipal landfill, and slow velocity.

A site visit was conducted in June 1990 to obtain pertinent information pertaining to the environment along the proposed channel. During the course of the site evaluation, soil resistivity measurements and water oxygen concentrations, pH and conductivity values were recorded on regular intervals along the river. In addition, soil and water samples were obtained at various locations. Soil samples were sent to a qualified laboratory where analysis was performed with respect to: conductivity, pH, chloride, sulfate, moisture content, redox potential, and total alkalinity. The constituents evaluated for the water samples included conductivity, pH, chlorides, sulfates, redox, total alkalinity and hardness.

An evaluation of the soil and water samples indicated that the environment to which the channel structures will be exposed is an extremely aggressive marine environment. The test results indicate that a salt water wedge is present at the bottom of the river along the entire section evaluated. This intrusion results in high concentrations of chlorides and sulfates in both the water and soil along the river.

The most important factors in achieving a desirable life for prestressed concrete are the quality of the concrete and the depth of cover over prestressing wires. The quality of the concrete is dependent on cement composition, aggregate size and purity, water quality and mix design. The depth of cover of the concrete over prestressing wires is critical to act as a buffer between the steel wires and the corrosive environment. In addition, by maintaining a

desirable depth of cover, the chance of exposing the prestressing wires by way of surface cracks in the concrete is limited.

Based on the test data accumulated, chlorides and sulfates are in sufficient concentrations to warrant concern over the deterioration of prestressed concrete piles proposed at the project site. Chlorides will penetrate the concrete and lead to corrosion on the prestressing wires. This would be most pronounced in the tidal zone where free oxygen accelerates the corrosion reactions. If left unchecked, spalling and eventual wall failures would result. Sulfates in the soil and water would directly attack the concrete, eventually leading to spalling. Sulfates also accelerate the attack of chloride on the prestressing wires by breaking down the concrete's ability to buffer.

To mitigate the corrosion activity, the prestressed piles should be of superior quality. Mixing water would be of potable quality. The use of sea water would prohibited. Portland cement would have a C3A content of 5 to 6 percent and alkali content of not more than 0.65 percent. Aggregates would be strong, durable, free from chlorides, nonalkali reactive, and graded to obtain dense concrete. Prestressing concrete would be free of contamination with a minimum depth of cover of 2 in. Epoxy coated strands would be used. The piles would be furnished with a factory applied coating. Once driven, all above grade portions of the concrete piles would be recoated to a point 6 in. below the mean low water line. This coating must be visually inspected and repaired on an annual basis.

The application of cathodic protection to control corrosion activity on prestressed concrete has been extremely limited due to the possible embrittlement of the wires by hydrogen generated at the cathode surface. Because of this complexity and the difficulty in maintaining proper protection levels, cathodic protection of the prestressed piles does not appear technically or economically feasible for this project.

Carbon steel sheet piles and associated tiebacks would be subject to severe corrosion attack. This attack would be most pronounced in the tidal zone and at the mud line due to oxygen differential concentrations.

The use of a good quality, factory applied coating would significantly reduce the extent of corrosion on the sheet piles. A 6-in. concrete facing for the steel sheet piling is proposed to provide a smoother surface needed for hydraulic requirements. This concrete would provided additional protection from the corrosive environment. The coating of any exposed portions of steel sheet pile would require regular maintenance.

An impressed current cathodic protection system would be a possible method to control corrosion on the submerged and buried portions of the sheet piles, however due to the extensive maintenance it would require, it is not recommended for this project.

Environmental Issues

The area of greatest ecological value within the basin impacted by the project is the area between De Diego Expressway and the joint outlet of the Rio Puerto Nuevo and Martin Pena into San Juan Harbor. The main areas of interest are the mangrove and wetland sectors along Rio Puerto Nuevo, Margarita, and Martin Pena, which cover some 115 hectares (284 acres). A total of 13.5 hectares (33.4 acres) of mangroves and other wetland vegetation would be removed as a result of the proposed plan of improvement. Replanting of mangroves along sectors of the improved Rio Puerto Nuevo and Margarita would be incorporated into a mitigation plan.

Material excavated from the lower portion of Rio Puerto Nuevo is scheduled for ocean disposal offshore in the existing designated Ocean Dredged Material Disposal for San Juan Harbor. The site is about a mile and a half offshore and in about 1,200 ft of water. The predominant portion of the material is

composed of undisturbed, saline, new work material. All of that new work material should be acceptable for ocean disposal. Bioassays and elutriate tests were performed on material from the existing Rio Puerto Nuevo and Margarita channels. Testing showed that material obtained from the existing Margarita upstream from its confluence with Rio Puerto Nuevo was unsuitable for ocean disposal. This would impact only a minor portion of the total material to be removed. Material unsuitable for ocean disposal would be placed in an upland disposal site.

Preliminary considerations of hazardous and toxic wastes involved the possibility of excavating material from the sanitary landfill adjacent to the lower reach of Rio Puerto Nuevo. The original channel alignment required excavation cutting through the landfill significantly at one bend in the alignment. Design changes were made that eliminated the necessity for this excavation.

The visual impact of the flood control project was an additional design consideration. Screening the channel through the use of berms, vegetation, and fencing materials will be done where possible to reduce its visibility and increase its aesthetical value.

The proposed improvements also include a bicycle path and linear park. It will be a major link in the overall system, ranging from the heart of Old San Juan to the proposed University of Puerto Botanical Gardens, being planned as part of the 500th anniversary celebration of the discovery of the Americas in 1992.

Conclusion

"Total quality design" was the primary focus throughout the initial planning and design process. This was achieved through intense coordination of the various technical aspects, as well as the environmental and local concerns. Value engineering was implemented as a continuous and dynamic process rather than something which occurred upon completion of contract plans and specifications as part of the review procedures. All too often "quick fixes" are employed when different elements are developed independently and then are found to be incompatible. By utilizing the "Total Quality Design" concept, a far superior product can be realized in a more cost effective and timely manner.

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Upper St. Johns River Basin Project Flood Control Structures

Charles B. McManus, PE[1]

Abstract

The Upper St. Johns River Basin is situated in east central Florida. The area includes much of Brevard and Indian River Counties and parts of Okeechobee, Orange, Oscelola, St. Lucie, Seminole, and Volusia Counties. The project consists of an approximate 1,780-square-mile area, bounded by the Atlantic Coastal ridge to the east and the Kissimmee River Basin to the West. The project involves the construction of levees, weirs, canal plugs, control structures, water management areas, and acquisition of land for flood control purposes which will also provide for water conservation and marsh enhancement. As part of this project, approximately 85 miles of levees, 4 reinforced concrete ogee weir spillways, 11 culverts structures, and 4 sheet-pile weirs are being constructed. The low ogee-type spillway with vertical lift gates has been model tested by the US Army Engineer Waterways Experiment Station and typically used by the Corps of Engineers in Florida since the early 1960's. The stilling basin for the low weir spillway consists of a horizontal apron with baffle blocks and an endsill with riprap being proved upstream and downstream of the structure to protect against erosion.

Introduction

The project area is located in east central Florida between the Florida Turnpike (south of Vero Beach) and US Highway 192 (west of Melbourne). It was originally a broad expanse of freshwater wetlands. Since the 1960's, significant portions of the area were developed for agricultural production particularly in Brevard and Indian River Counties. Portions of the eastern and western sides of the river valley were diked and much of the former drainage into the St. Johns River Valley from the east was redirected to the Indian River. Following storm events, some agricultural interests pump flood waters from agricultural lands into the St. Johns River. The Upper St. Johns River Basin Project provides marsh conservation areas and water conservation areas to separate agricultural waters from the historic St. Johns River. The flood control structures, being constructed as a part of this project, will be operated to more naturally control the flow in the river.

Authority

Flood control features for the Upper St. Johns River Basin were originally authorized by the Flood Control Act approved September 1954. The current plan was approved September 1986 in the Central and Southern Florida Project General Design Memorandum, Part 3, Supplement 2, Addendum 3 (Us Army Engineer District, Jacksonville, 1984). The Local Cooperation Agreement was signed in

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December 1987, and the St. Johns River Water Management District (the local sponsor) has agreed to provide, without cost to the US, all lands and right-of-way necessary for construction of this project.

Project Description

The project involves the construction of approximately 85 miles of levees, 4 ogee weir spillways, 11 control structures, several canal plugs, 2 water management areas, and acquisition of land for purposed flood control, which also can be used for water conservation and marsh enhancement. The objectives of the Upper St. Johns Project is to provide flood protection, preserve and enhance the existing flood plain, improve water quality, ensure adequate surface water supply and protect cultural resources. The project is designed to contain floodwaters of the Upper St. Johns River, restore and preserve freshwater marshes in the river valley, and protect water quality by limiting agricultural runoff into the Upper St. Johns River. The selected plan provides the most economic benefits of all plans considered. When completed, the plan will consist of the Jane Green Flood Detention Area; the Taylor Creek Reservoir; the St. Johns and Blue Cypress Water Management Areas (designed to contain agricultural runoff and flood waters from tributary areas to the east); The Blue Cypress and Fort Drum Marsh Conservation Areas (designed to restore and preserve historic freshwater wetlands in the river valley); and levees along sections of the eastern and western river valley to contain floodwaters and prevent flood damages. The plan contributes to the national, regional, and local economy while providing environmental enhancement of existing areas and returning developed and semideveloped areas to their wetland state.

Description Of Structures

Structure 161A is the largest structure being constructed as a part of this project. Structure 161A is a two-bay, reinforced concrete ogee weir, gated spillway. The vertical-lift gates are manually controlled. The structure has a 13-ft, 8-in.-wide reinforced concrete service bridge, a reinforced concrete operating platform, steel sheet-pile wingwalls, and a control house. To control the seepage below the structure, a steel sheet-pile cutoff wall is provided under the upstream and downstream edge of the foundation slab of the structure. The structure is constructed with a needle beam recess upstream of the vertical-lift gates so that the structure can be dewatered by the use of needles and needle beams. Such a closure serves not only for maintenance but also as an emergency temporary closure, if a gate needs to be removed. Riprap is provided upstream and downstream of the structure to protect against erosion. Safety/debris barriers have been provided upstream and downstream of the structure. The structure is designed to reduce high river stages in the St. Johns River. This is possible by holding floodwaters upstream of S-161 in the Jane Green Detention Area and by making releases over a longer period of time with discharges being made as required to maintain design stages in the St. Johns flood plain downstream of the structures. Since the operation of the structure does not call for increasing the stages in the creeks and lowlands in Jane Green Swamp downstream of the structure, S-161 has been constructed with two diversion culverts located in the sidewalls of the spillway to maintain low-flow discharges. These culverts provide for passage of flow at stages lower than the crest elevation and provide a means for a complete drawdown of the reservoir upstream of the spillway.

Spillway Structure 161A	
Crest Elevation	26.0 ft
Vertical-Lift Gates	Two Gates (21.3 by 16.2 ft)
Discharge Capacity	8,465 cfs
Cost	$3,364,894

Structure 96B is a single-bay, reinforced concrete, ogee weir, gated spillway. The vertical-lift gates are manually controlled. The structure would include a 13-ft, 8-in.-wide reinforced concrete service bridge, a reinforced concrete operating platform, steel sheet-pile wingwalls, and control house. A sheet-pile cutoff wall is provided under the upstream edge of the structure and the structure is designed to

be dewatered by the use of needles and needle beams upstream and down stream of the vertical-lift gate. This closure would serve not only for maintenance, but also as an emergency temporary closure if the gate needs to be removed. Riprap is provided upstream and downstream of the structure to protect against erosion. Safety barriers are provided upstream and downstream of the structure. S-96B is the main outlet for the St. Johns Water Management Area (SJWMA), allows discharges to be made into the historic flood plain of the St. Johns River, and will supplement the discharge capacity of S-96.

Spillway Structure 96B	
Crest Elevation	13.1 ft
Vertical-Lift Gates	One Gate (20.0 by 10.9 ft)
Discharge Capacity	1,000 cfs
Cost	$1,428,737

Structure 96C is a single-bay, reinforced concrete spillway provided with manually controlled vertical-lift gate installed on the crest of the ogee weir. This structure is provided with a reinforced concrete operating platform to accommodate the gate operating equipment and a reinforced concrete service bridge that would provide vehicular access across the structure. S-96D would replace the existing steel sheet-pile structure (S-1) and control flow from the Blue Cypress Marsh Conservation Area (BCMCA).

Spillway Structure 96C	
Crest Elevation	11.8ft
Vertical-Lift Gates	One Gate (25.0 by 13.7 ft)
Discharge Capacity	1,500 cfs
Cost	$1,607,000

Structure 96D is single-bay, reinforced concrete ogee weir, gated spillway structure constructed with a reinforced concrete operating platform and reinforced concrete service bridge. Safety barriers were provided upstream and downstream of the structure with riprap protection being provided upstream and downstream of the structure to protect against erosion. Due to the anticipated settlement of the foundation, this structure was constructed on a pile foundation. S-96C is the main outlet form Canal 65 from the Blue Cypress Water Management Area (BCWMA).

Spillway Structure 96D	
Crest Elevation	15.5ft
Vertical-Lift Gates	One Gate (15.0 by 12.1 ft)
Discharge Capacity	1,000 cfs
Cost	$964,558

Other structures being constructed as a part of the Upper St. Johns Project are S-250 A, B, and C (three structures each with a single 72-in.-diam culvert adjacent to a 135-ft sheet-pile weir), S-251 (four 72-in.-diam culverts with slide gates), S-252A (two 60-in.-diam culverts with slide gates), S-252B (two 60-in.-diam culverts), S-252C (one 60-in.-diam culvert), S-254 (1,500-ft concrete sheet-pile weir), S-255 (four 72-in.-diam culverts with flap gates), S-256 (three 72-in.-diam culverts with flap gates), S-257 (one 72-in.-diam culvert with slide gates). All culverts are being constructed with safety barriers upstream and downstream. To protect against erosion, riprap or fabric form slope protection is being provided upstream and downstream of each structure. Culverts with slide gates were provided with an access walkway and operating platform with manually operated gear lifts for the gates.

Design Considerations

Structures 96B, 96C, and 96D are single monolith structures. In general, these structures were analyzed for three loading conditions: Construction Condition (Case I), with no hydrostatic forces acting; Dewatered Condition (Case II), with water surfaces at the optimum levels; and Operating Condition (Case III), with the maximum differential in head conditions. Select structural backfill was used for the foundation for S-96B and S-96C, as a layer of peat and clay, existing immediately below the structure, was removed. Due to anticipated settlement of the interbedded clay strata at S-96D, this structure was constructed on a timber pile foundation. All of these structures are very stable for all loading conditions. The resultant of the load is always within the Kern area.

Structure 161A is separated into three monoliths (gate control monolith and two stilling basin monoliths). This structure was analyzed for four loading conditions: Construction Condition (Case I), with the structure complete, no backfill or hydrostatic loads; Construction Condition (Case II), with the structure complete and backfill in place; Operating Condition (Case III), with normal water levels; and S.P.F. Flood Condition (Case IV), with gates fully open and hydraulic jump occurring. A highly compressible layer of clay and clayey sand was removed and replaced with compacted granular material (select structural backfill) to form the foundation for the structure. The large lateral water loads acting on the control monolith during Standard Project Flood Condition necessitated that extension of the base slab be added on each side of the structure and that an uplift relief system be provided under the control monolith.

Construction Status

The Upper St. Johns Project consists of 12 contracts, 7 of which have been awarded to date. One contract is scheduled to be awarded in FY-91, two contracts scheduled to be awarded in FY-92, and two contracts are scheduled to be awarded in FY-93. The total cost of the project is estimated at $165,235,000. The project is anticipated to be complete by FY-95.

Penetration Resistance and Blast Response of Masonry Walls

by
David R. Coltharp[1]

Abstract

The penetration resistance and blast response of masonry walls are of interest to engineers faced with designing masonry structures to resist the effects of accidental explosions and/or terrorist weapons (such as rockets and vehicle bombs). While procedures and data are available for the design of reinforced concrete facilities to withstand these threats (Department of the Army 1969, 1986), similar information is lacking or insufficient for masonry construction. This paper summarizes recent research to advance the state of the art in this area. The following topics are discussed: (a) the resistance of masonry walls to small arms penetration, (b) protection of masonry walls from shoulder-fired rockets, and (c) the blast response of masonry walls.

Penetration Resistance to Small Arms

Ballistic-resistant building components are designed to prevent complete penetration of the projectile and protect a person standing directly behind the barrier from injury. In general, ballistic-resistant building components are those that meet the testing requirements of certain industry and/or government standards. These standards are generally limited to the category of firearms denoted as "small arms" and refer to specific military and civilian issue weapons that can be fired from the hand or shoulder. The most common of these standards are the American National Standards Institute/Underwriters Lab, Inc. (1988) and the National Institute of Justice (1985). Both identify different levels of threat weapons ranging from low velocity handguns (e.g., .357 Magnum and .44 Magnum, 1000-1500 ft/sec) to higher velocity (2500-3000 ft/sec) ball and armor-piercing (AP) projectiles (e.g., 7.62mm M80 Ball and 7.62mm M61 AP). Because of the wide variety of small arm weapons and projectile types, the differences in materials, and the ease of conducting tests, most information on the penetration resistance of materials is empirical.

While there is a wealth of data on the ballistic resistance of building components such as window glazing material and various types of doors, there are surprisingly little data available on walls, specifically masonry walls (Marchand 1990). Data from several sources (US Army Corps of Engineers 1990, Department of the Navy 1983, Goddard 1969, and

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Ball 1990) were used to generate the bar graph in Figure 1 to give the reader an indication of the types of masonry construction that will offer protection from various levels of threats. As seen from the graph, even a single layer (4 in.) of brick will defeat the low to medium level threats and a 12 in. brick wall will stop the higher threats. Note that the graph is based on "single-hit" data, i.e. where 1-5 rounds of the threat weapon were fired against the target in a closely spaced grouping but such that each penetration was independent of the others.

Extrapolation of this information to other walls or weapons should be done with caution especially when extrapolating from the relatively soft lead-core ball ammunition (such as the 7.62mm M80 Ball which deforms significantly during penetration, see Figure 2) to the stronger hard steel and tungsten cored ammunition (such as the 7.62mm M61 AP projectile which has a kinetic energy similar to the M80 Ball round but retains its shape after initially shedding its copper jacket during penetration, see Figure 3).

Protection from Shoulder-Fired Rockets

Shoulder-fired rockets are military issue weapons that are primarily designed for defeating armored vehicles such as tanks. They usually rely on an explosive warhead with a conical-shaped metal liner that collapses on detonation of the explosive and forms a high velocity (typically 20,000 ft/sec) molten "jet" of metal that can penetrate over 12 in. of steel. Typical weapons in this category include the US Light Antitank Weapon (LAW) shown in Figure 4, the Soviet Rocket-Propelled Grenade (RPG-7, etc.) shown in Figure 5, and the Swedish Carl Gustaff (84mm High Explosive Anti-Tank - HEAT). These weapons can be operated by one person and are accurate up to a few hundred yards. Because of their antitank capability they can penetrate several feet of concrete or masonry material and it is generally not practical to design masonry buildings with walls thick enough to directly defeat these weapons.

One approach to this problem is to use opaque screens of either wood, sheet steel, or similar material in front of the wall to be protected (Figure 6). This accomplishes two things—it hides the target from view since shoulder-fired rockets are line-of-sight weapons, and the screen material will cause the sensitive fuse on the nose of the antitank rocket to function and detonate the warhead before it impacts the target. Tests by the Waterways Experiment Station (WES) and others have shown that if sufficient standoff is provided between the "detonation" screen and the wall, the molten jet of metal will disperse and solidify and penetration of the shaped charge "jet" will not occur. The minimum standoff distance required for defeat of these rounds depends on the specifics of the weapon system and the construction of the masonry wall and is determined experimentally. Typical distances are on the order of 10- to 40-ft.

Newer shoulder-fired rockets are becoming available to military forces which are designed for fighting in cities and are made specifically for breaching masonry and concrete walls of buildings. The WES is currently developing methods for defeating these weapons. Results should be available in September 1991.

Blast Response

The blast response of masonry walls is important for predicting the damage to existing structures from accidental explosions or designing buildings to resist blast loads. Accurate prediction of the blast response is complicated by the uncertainty in accurately knowing the blast loads, the wall support conditions, and the wall material properties. Sophisticated finite element computer codes can be used to calculate the dynamic response of the wall under blast loading, but, given the degree of uncertainties mentioned above, it is usually sufficient to approximate the structural response using a simplified single-degree-of-freedom (SDOF) model (Biggs 1964). Weihle and Bockholt (1968, 1970), and Weihle (1974) describe such a procedure for long duration nuclear blast loads on a masonry structure.

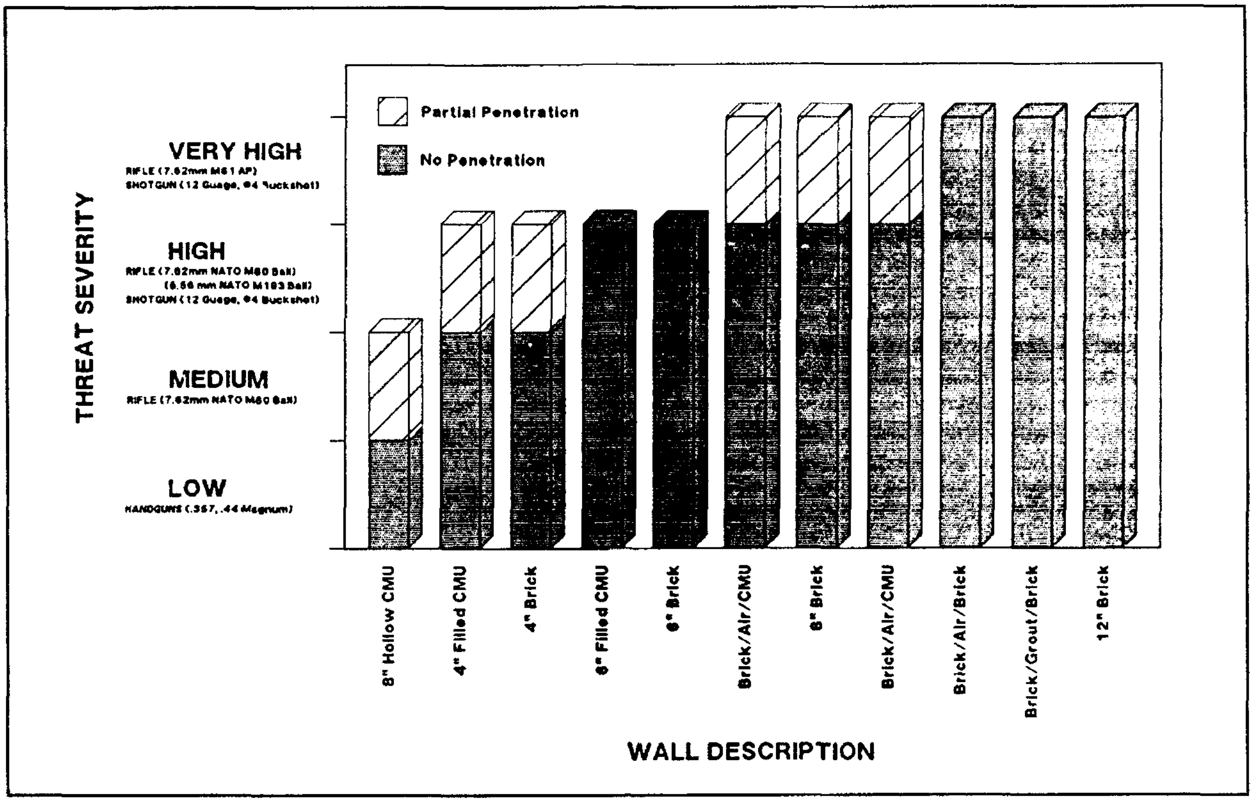

Figure 1. Ballistic resistance of masonry walls (Based on multiple sources)

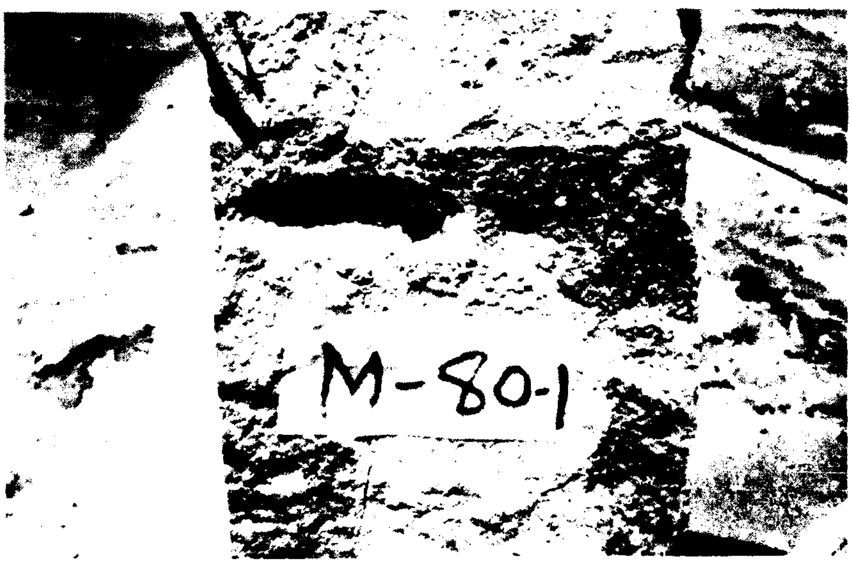

Figure 2. Deformation of 7.62m M80 lead core projectile on penetration of 8 in. grout-filled concrete masonry unit (CMU) wall

Figure 3. Deformation of 7.62m M61 AP projectile into 8 in. grout-filled CMU wall. (Note position of copper jacket shed during penetration process and position of hard steel core.)

Figure 4. Firing of the US Light Antitank Weapon (LAW)

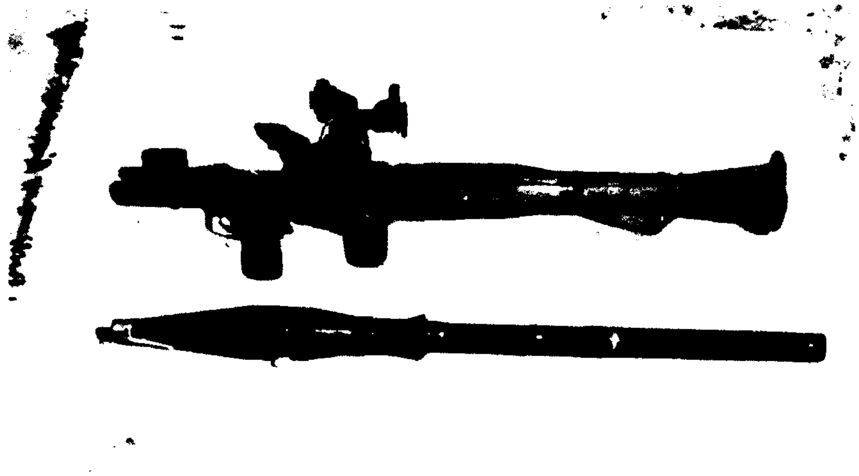

Figure 5. Soviet RPG-7 Shoulder-Fired Rocket System

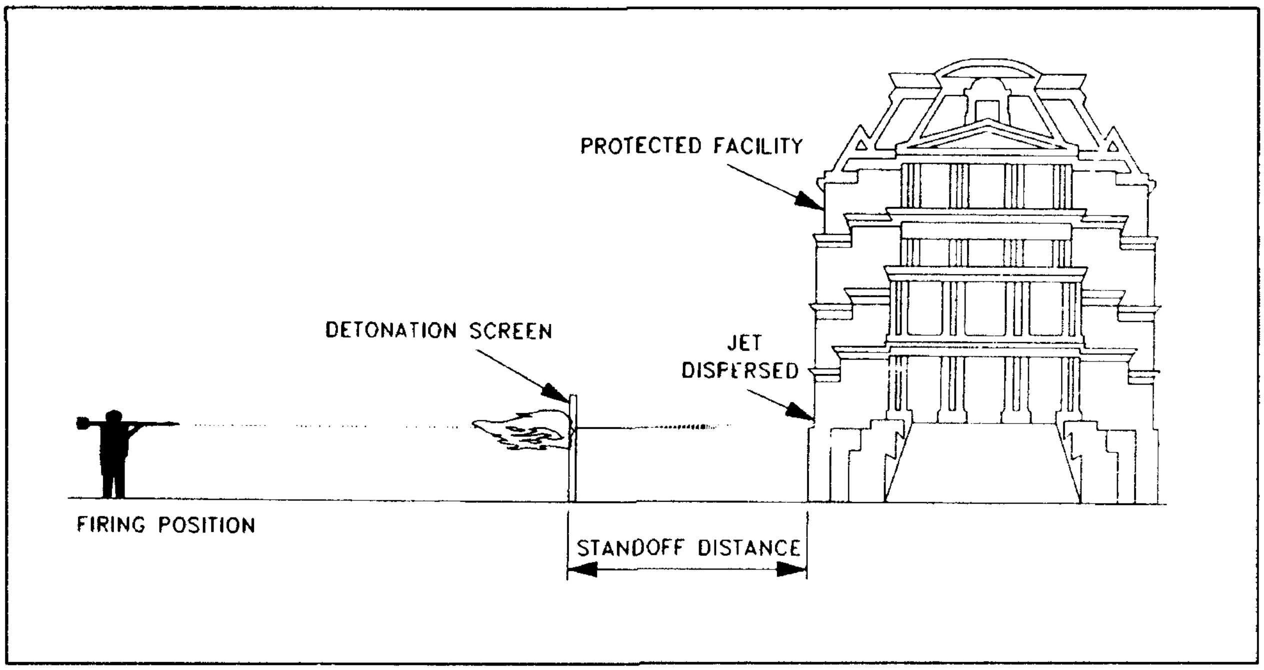

Figure 6. Protection of a facilityu using a detonation screen

The WES has updated this SDOF procedure with current blast load prediction equations for conventional explosions.

Figure 7 depicts how the real wall system is modeled as a simple SDOF spring and mass system. The equation of motion of the idealized system can be written as

$$M_e\, y'' + R_e(y) = F_e(t)$$

where M_e, $R_e(y)$, and $F_e(t)$ are the equivalent mass, resistance, and load for the SDOF system, respectively. This equation can also be written as

$$K_{lm}\, M_t\, y'' + R(y) = F_t(t)$$

where M_t, $R(y)$, and $F_t(t)$ are the mass, resistance, and load associated with the actual wall, respectively. K_{lm} is the load-mass transformation factor and depends on the assumed deflected shape of the wall and the load distribution. Tabulated values for K_{lm} for a variety of support conditions and load distributions are given in Biggs (1964). In addition to the load-mass transformation factor, the additional parameters required for the solution of the equation are the resistance function and the load-time function.

The resistance function is defined as the maximum deflection of the wall as a function of the imposed load on the wall. The resistance function can be developed from static load considerations or tests as long as the static deflected shape and load distribution are the same as the dynamic ones.

For unreinforced masonry walls with simple or fixed supports, the resistance function is similar to that shown in Figure 8. There is an initial elastic phase followed by cracking of the mortar and formation of plastic hinges. Resistance following hinge formation is due to the restoring moment developed by the eccentricity of the weight of the wall (Figure 9). Under a static load condition, the decaying phase of the resistance function is unimportant since collapse of the wall will occur when the load reaches the value of the peak resistance. For a short duration blast load, however, this phase significantly influences the blast capacity since plastic hinge formation occurs at very small deflections and absorbs very little energy compared with later decaying phase of the walls resistance. For longer duration blast loads, this effect would not be as noticeable.

If masonry walls are used as infill panels of a steel or reinforced concrete frame building and if there is no gap between the top of the wall and the upper concrete or steel beam, then the wall will behave as a three-hinged arch under lateral loading, and will have a resistance function similar to that shown in Figure 10. As the wall begins to rotate under loading, the interior top and bottom edges are prevented from moving up or down by the rigid beams. This gives rise to compressive inplane forces in the wall which significantly increases its resistance. As the load increases, the edges are crushed and a hinge forms at the middle of the wall and the resistance begins to decay. In general, "arching" walls are significantly stronger than walls with simple or fixed supports and can approach strengths typical of reinforced masonry.

For reinforced masonry, the resistance function is similar to that shown in Figure 11 where there is an initial rise until cracking of the masonry, then a slower rise until the steel yields and hinges are formed, and then a constant phase as the steel undergoes plastic deformation.

A comparison of resistance functions for different types of solid masonry walls is shown in Figure 12. A comparison of the walls blast load capacity is given in Table 1 for the case of an explosion occurring at a distance of 350 ft from the wall, as denoted in Figure 12.

For comparison, the Department of Defense (1978) explosive safety standards gives 1,000 pounds as the maximum permissible explosive weight that can be stored in a covered

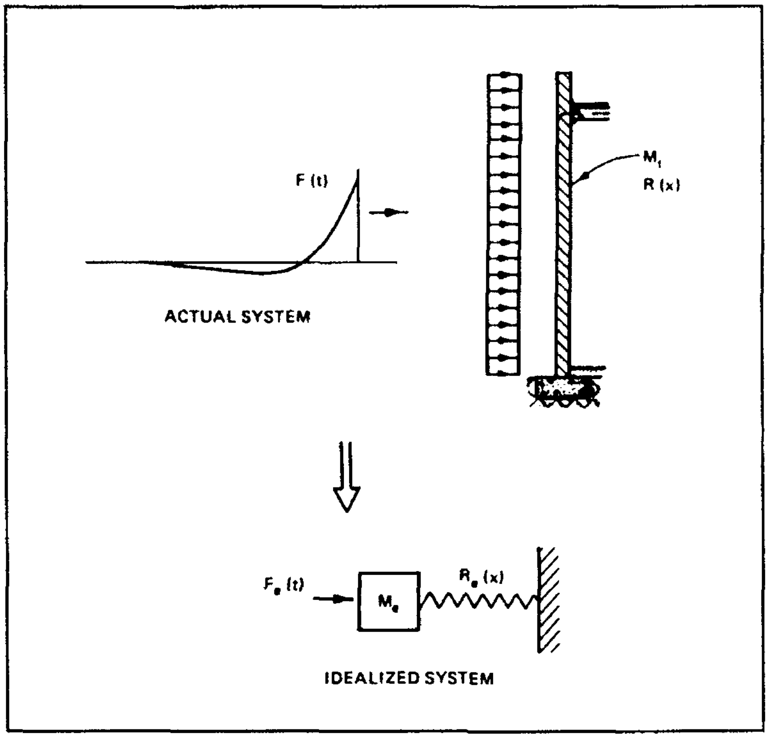

Figure 7. Single-degree-of-freedom idealization for blast response of ma^onry walls

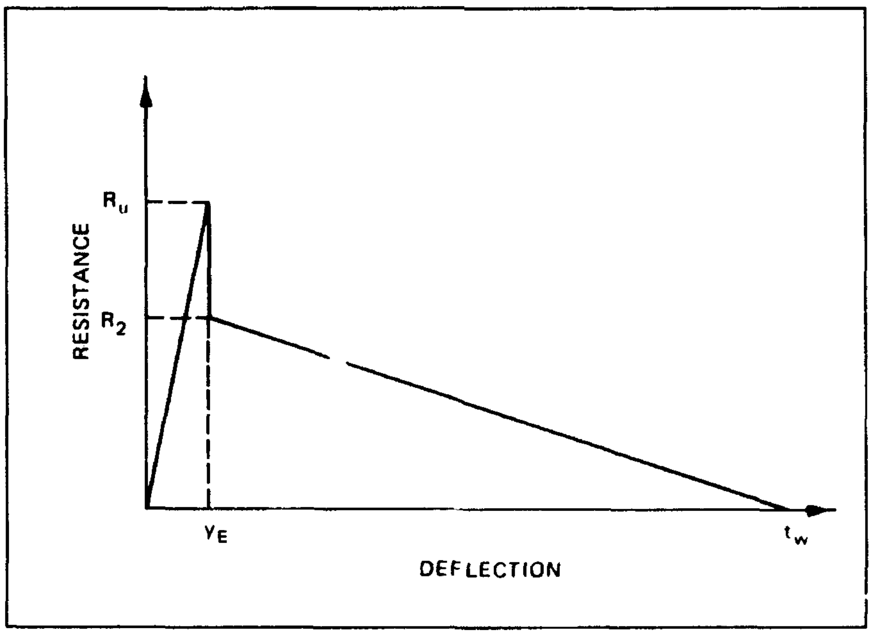

Figure 8. Typical resistance function for unreinforced masonry walls

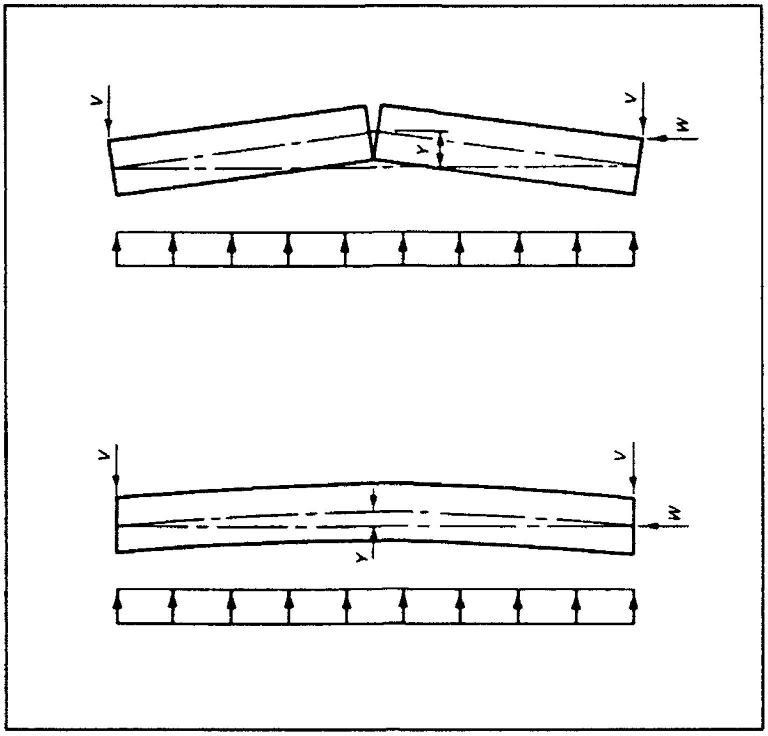

Figure 9. Assumed response of unreinforced masonry wall to uniform load

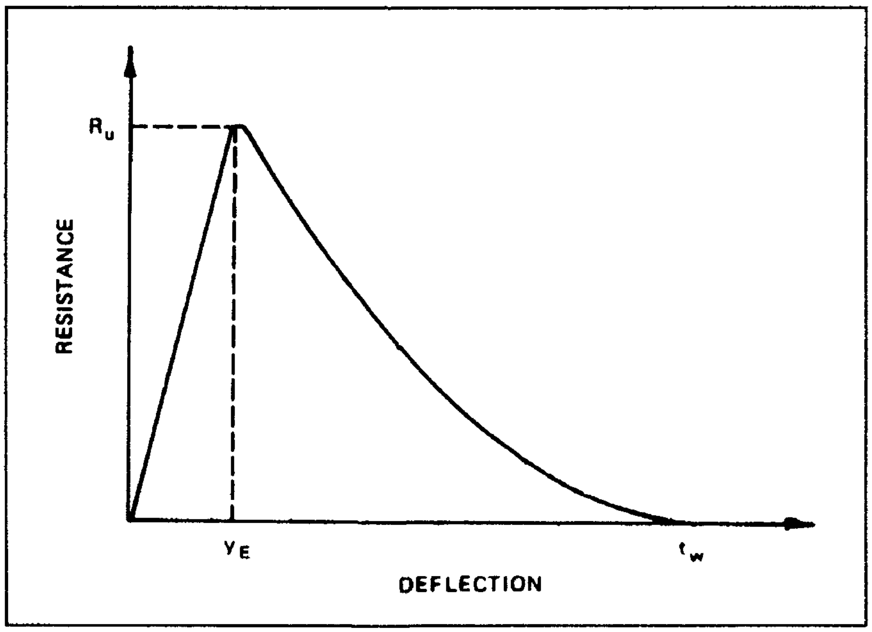

Figure 10. Typical resistance function for masonry wall assuming arching action

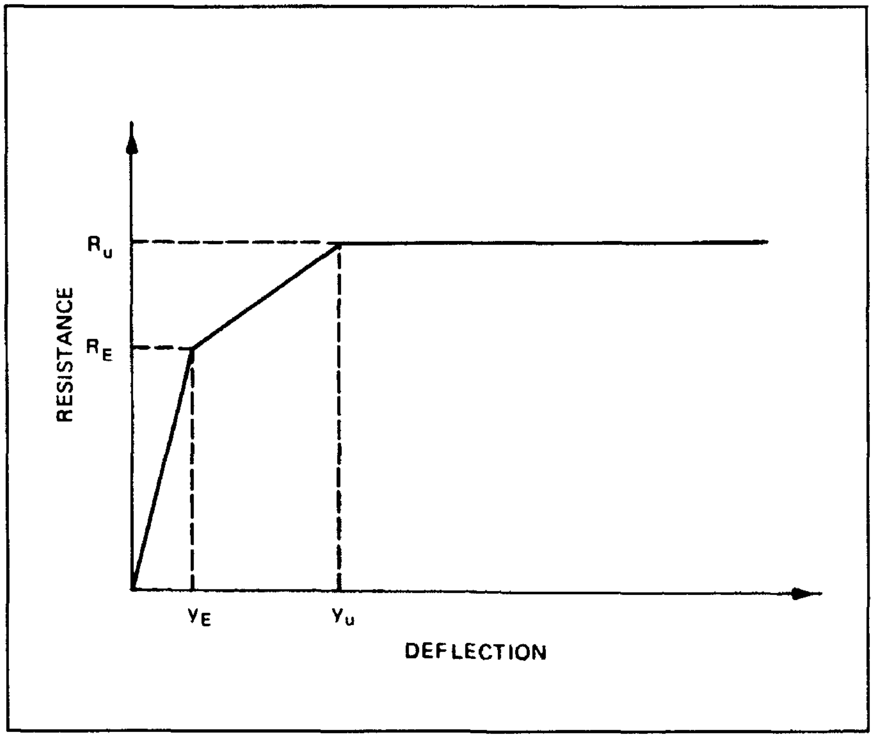

Figure 11. Typical resistance function for reinforced masonry wall

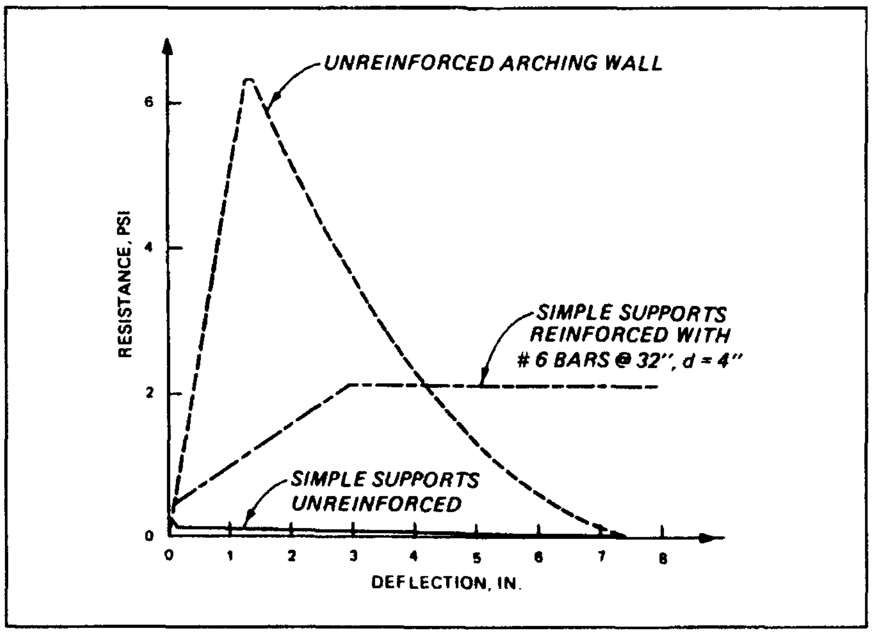

Figure 12. Comparison of resistance functions for 8-in. CMU walls, fully grouted, 12-ft spans

Table 1
Comparison of Blast Capacity of Masonry Walls

Wall Type	Equivalent TNT Explosive Weight (lb) at a Distance of 350 ft for		
	Light Damage[1]	Medium Damage[2]	Wall Collapse[3]
Simple supports, unreinforced	110	160	760
Reinforced with #6 bars @ 32 in. in center of wall	590	1,200	6,830
Unreinforced infill "arching" wall	1,200	3,920	11,700

Notes: Walls analyzed were: One-Way, 12-ft spans, 8 in. thick fully grouted CMU
1 Light Damage refers to a wall rotation of 0.5 deg, wall will suffer only light cracks and be reusable.
2 Medium Damage refers to a wall rotation of 1.0 deg, wall will not collapse but will need replacing.
3 Collapse refers to a deflection equal to the wall thickness, wall will suffer heavy damage to collapse. Occupants would be in danger.

storage magazine at this distance from an inhabited building. The weights given in Table 1 were calculated using a blast load function similar to that shown in Figure 13. Note that it has both a positive and negative phase. Experiments, calculations, and actual incidents indicate that for some masonry walls an interesting phenomenon occurs where the negative or "suction" phase of the loading causes failure and the wall collapses to the outside of the structure.

Another interesting phenomenon occurs for walls with openings such as "soft" windows and for walls with blast-resistant or "hard" windows. A soft window is quickly blown away and the reflected blast pressure on the wall near the open hole is relieved by air flow through the opening. This results in less load on the surrounding wall and thus less structural damage. In the case of a blast-resistant window, this pressure relief doesn't occur. In addition, the load from the reflected blast pressure acting on the window is transferred to the wall at the window supports as a line load. This can result in a wall with a blast-resistant window system having less blast capacity than either a solid wall or a wall with a "soft" window. Figure 14 and Table 2 illustrate this effect for a particular wall and for an explosion occurring at a distance of 300 feet away.

From a design standpoint, the conclusion is that blast-resistant windows and walls should be designed as a system rather than independently and that extreme care should be taken when retrofitting a masonry structure with blast-resistant windows. In some cases, it may be better to provide support for blast-resistant windows at the floor and ceiling instead of the wall.

Table 2
Comparison of Blast Capacity of Walls with Windows

Wall Type	Equivalent TNT Explosive Weights (lb) at a Distance of 300 ft for Collapse of Wall Shown in Figure 14
Solid wall	20,000
Wall with soft windows	33,800
Wall with hard windows	18,000

Conclusion

Although procedures for the design of penetration and blast resistant masonry structures are not as advanced as for reinforced concrete structures, there is information available from recent research to aid the designer. More research is needed and is planned for the protection of masonry walls from other shoulder-fired rockets, and for the blast resistance of cavity wall construction and walls with openings to improve current analysis methods.

Acknowledgements

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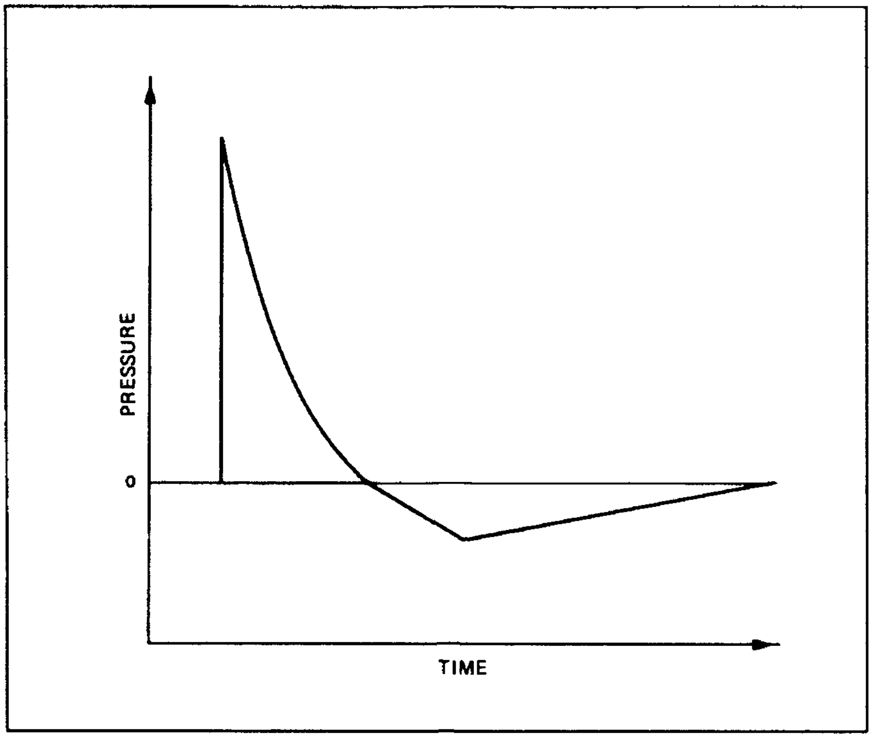

Figure 13. Idealized pressure-time history for short duration blast load

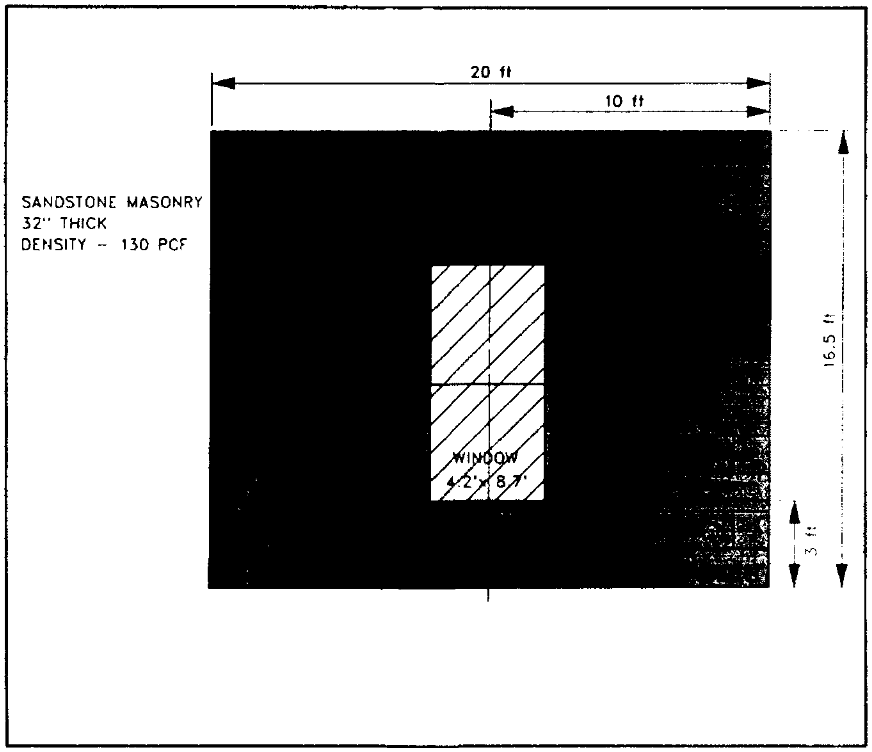

Figure 14. Masonry wall with window opening

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Nonlinear Finite Element Analysis Applied to Blast-Resistant Structures

by
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Abstract

The tools available to the designer for analyzing the nonlinear behavior of blast loaded above ground and below ground protective structures are changing. This paper contains a discussion of the single-degree-of-freedom (SDOF) method and the finite element method for modeling structures and presents some of the advantages and disadvantages of each method. Structural analysis examples of protective structures that demonstrate when each method is appropriate are provided. These examples include finite element analyses of an earth-mounded reinforced concrete arch, a large multistory semihardened facility, and a reinforced concrete deep beam. Standard methods for calculating the ground shock loads to be applied to the structural models are also discussed and illustrated in the examples.

Introduction

The design of protective structures to resist conventional weapons has changed during the past few years. The SDOF models for the design of flexural elements of structures and for determining structural response for in-structure shock are being supplemented by the use of two-dimensional and three-dimensional multiple-degree-of-freedom models. Also, the simplified methods for calculating ground shock loadings from buried bursts are being supplemented by more automated and sophisticated methods. Each level of model complexity for calculating dynamic response has advantages and disadvantages, and the choice of which model to use is not always clear. This paper attempts to illustrate when the use of the more detailed analysis procedures is justified by describing the limitations of the SDOF method and by discussing actual structural analysis examples that use the finite element method.

Single-Degree-of-Freedom Analysis

In SDOF analysis, the behavior of a structural element is represented by a single spring-mass system acted upon by a time-dependent load. The deflection of the spring-mass system is set equal to the maximum deflection of the structural element. The spring-mass system is made "equivalent" to the structural element by means of transformation factors applied to the mass and load; i.e. mass and load factors. For a simple undamped system, these two factors can be combined into one load-mass factor. The resistance of the structural element can be modeled well into the plastic range.

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SDOF analysis criteria first appeared in the Technical Manual TM 5-856-1 (Headquarters, Department of the Army, 1959) series for nuclear weapon resistant design in the 1965 supplement. The method was later adopted for conventional weapons resistant design in TM 5-855-1, (Headquarters, Department of the Army, 1986) and is a cornerstone in the revisions for TM 5-1300, (Headquarters, Department of the Army, revisions in preparation) for design of structures to resist the effects of accidental explosions. Recent innovations in the SDOF method include:

- The addition of a soil structure interaction algorithm for the analysis of below-grade walls subjected to ground shock.
- The inclusion of a second degree of freedom to account for element support motion; i.e., rigid body structure motion.
- The inclusion of compression and tension membrane behavior in one- and two-way concrete slab systems.

The SDOF method has been an accurate and practical tool for the design of many structures subjected to time dependent loads. However, the method is limited in application. Some of these limitations are described:

- The SDOF method tends to be restricted to simple geometries, such as one- and two-way slabs, although some formulations have been developed for more complex shapes such as semicircular arches.
- When used for the analysis of one- and two-way slabs, the SDOF method is most suitable for slabs having a span-to-depth ratio greater than five and breaks down completely when the span-to-depth ratio approaches three. This is because most SDOF models ignore shear deformations and assume point or line hinge mechanisms in the plastic range. Deep members do not behave in this manner.
- Flexure is predetermined to be the mode of failure in SDOF analysis. Therefore, other failure mechanisms such as direct shear, diagonal shear, spall, and breach cannot be modeled.
- Only single elements can be modeled. This leaves open the possibility of failure due to instability of the overall structure.

Structure Motions

The calculation of structure motions is an important part of determining the shock environment inside structures subjected to loading from nearby explosions. Simple approaches to this problem include the following:

- The free-field modification method in which average free-field ground motions are adjusted to obtain structure motions.
- The SDOF and limited multidegree-of-freedom methods in which the structure is treated as a rigid body.

These methods of determining structure motion are most suitable for cases in which the overall structure translation dominates structure motion. These methods are not suitable for determining the motions of flexible structures in which the local member response dominates or in cases where a portion of the structure experiences motion while the remainder of the structure is nearly stationary.

Finite Element Analysis

Nonlinear behavior

The recent advances in nonlinear finite element analysis have made possible the solution of many problems that are not approachable by the more traditional and simple methods of analysis described earlier. These finite element methods take into account the effects of both large displacement and nonlinear material behavior. The structure can be treated as a continuum, and effects, such as concrete tension cracking, concrete crushing, and steel yielding and strain hardening can be captured without predisposition of the failure mechanism.

Soil-structure interaction

The behavior of below-grade structures subjected to ground shock loading is one of the most important problems encountered in conventional weapons resistant design. In finite element analysis, the following two approaches are commonly used.

- **Soil island.** In the soil-island method, both the soil and the structure are modeled in one finite element grid. The grid boundaries are generally loaded using the free-field ground shock velocities. Ground shock in the soil surrounding the structure, load transfer to the structure, and structure response are all calculated simultaneously. When properly applied, soil-island analysis comes very close to simulating the true event and provides detail behaviors both in the structure and the soil. The difficulty with this is that detailed constitutive soil properties must be obtained prior to the analysis.

- **Decoupled analysis.** In the decoupled analysis, the structure is modeled by a finite element grid and the soil structure interaction is modeled through an algorithm that accounts for the relative velocities of the soil and the structure. The interaction algorithm can be formulated using simple soil properties, and therefore less detailed soil property information is required. The finite element grid contains less nodes than required in soil-island analysis which reduces the time re.uired for computation.

Structural Analysis Examples

Aircraft shelter design

The aircraft shelter structure consists of an earth-mounded semicircular reinforced concrete arch. The arch mound is overlayed by a thick, concrete burster slab to prevent penetration by munitions.

A series of parametric analyses were performed to determine the optimum depth of earth cover, arch thickness, and arch reinforcement. These parametric analyses were performed using a decoupled two-dimensional model built up from beam elements that model nonlinear material behavior. Ground shock loads from weapon detonations were computed using the methods in TM 5-855-1 and accounted for only the direct blast load to each node.

Once the preliminary analysis was completed, a more detailed nonlinear finite element analysis of the arch was performed using the general-purpose computer program, ADINA (Bathe 1987). The arch concrete was modeled using rectangular plane strain elements and a concrete material model that simulates both tension cracking and crushing. Steel reinforcement bars were modeled discretely using line elements and a steel material model that simulates steel yield. This analysis provided more detail on the extent of concrete cracking and crushing behavior along the arch.

The blast load was calculated more accurately for the more detailed analysis by including multiply reflected shock waves from oblique reflecting surfaces. The more detailed model also allowed a static analysis of the arch to be followed by a dynamic analysis that included the gravity as well as blast loads.

A finite element analysis was chosen to model this arch because of the complexity of the blast loading and because the design manuals do not list mass and load factors for the SDOF analysis of arches. Mass and load factors have been derived for certain load cases for arches; but these have not been extensively validated and are not readily available to designers.

Multistory Semihardened Facility

The semihardened facility was 120 by 120 ft in plan with two stories above ground and a full basement. The structure was designed to resist the blast loads from the aboveground and below ground detonations of conventional weapons.

The various possible nonlinear analysis procedures and modeling options were evaluated to determine which was appropriate for this project. It was decided that the individual flexural elements would be analyzed using SDOF models and that the structural motion would be analyzed using a decoupled three-dimensional finite element model.

The airblast and ground shock loadings were computed by separate in-house computer programs and applied as input to the structural models. The load generation programs generate the loading waveforms using the equations in the current Army and Air Force manuals. They are capable of predicting the attenuation of direct as well as multiply reflected shock waves in three-dimensional space. Pressure time averaging was used over a grid of points for all wall and slab panels analyzed.

The flexural elements of the structure were analyzed for response to dynamic blast loadings using SDOF modeling. The elements analyzed by this method consist of all the exterior walls and slabs. The in-house SDOF computer program used for this analysis includes soil-structure interaction effects and is capable of direct input of the pressure-time loadings generated by the airblast and ground shock computer programs. This allows the actual complex loading histories to be used for design, eliminating the inaccuracies of the simplified triangular loading functions more commonly used in blast design.

The design of interior elements of the structure including columns, slabs, and beams was based on the application of pseudostatic reactions from the exterior elements. The pseudostatic reactions were calculated from the peak dynamic responses of the exterior elements.

A decoupled three-dimensional finite element model was chosen to model the structural motion of the facility for two reasons. The first is that the facility contained sensitive electronic equipment throughout its interior which required that an accurate in-structure shock and shock isolation analysis be performed. The second is that a large multistory facility like this does not respond as a rigid body. It flexes along its longitudinal and transverse axis as well as having localized flexure in the slabs between column and wall support points.

The finite element model more accurately models these higher order structural effects as the building responds to the highly localized dynamic blast loading. The model grid was intentionally made fairly coarse to shorten execution time. When only structure motions are required, a fairly coarse finite element grid can be used without appreciable loss in accuracy because displacement continuity is maintained across the nodal points. However, it must be recognized that coarse grid models depart from representing the structure as a continuum, and thus some behavior details are lost.

This model was built up using shell elements to simulate floor, wall, and roof slabs and beam elements to simulate interior columns. Decoupling the soil and omitting the soil finite element grid reduced model complexity and analysis time.

For this structure, where the structure model is loaded by conventional weapon induced ground shock, nonlinear structural behavior is present only in elements very close to the detonation. Thus, nonlinear plastic elements were used only near the detonation.

For this project, the SDOF flexural analysis of a single member took approximately 1 manday to formulate and run; and the three-dimensional structural motion analysis of the entire facility took approximately 1 man-month. This illustrates that, unless there is a specific requirement, the simple models should be used.

Deep Beam Analysis

The deep beam analyzed supports the vertical edges of a blast door subjected to airblast from an accidental explosion. The support member is 12 ft high, 6 ft deep, and 3.5 ft thick. The member is located inside the structure and is loaded both laterally and axially.

A detailed nonlinear finite element analysis was performed using the general-purpose computer program, ADINA. The structure modeled consisted of the deep beam and portions of the supporting floor and roof. The concrete beam was modeled using rectangular plane stress elements the floor and roof were modeled using rectangular plane strain elements and the steel reinforcement was modeled using line elements. The material models allowed for concrete tension cracking, concrete crushing, and steel yield. This analysis provided accurate deflection predictions at the blast door and concrete interface and a more accurate determination of the required reinforcement.

It was decided to study the deep beam using a finite element analysis because of the substantial axial load that was applied dynamically along with the flexural load. SDOF procedures do not model deep beams well because deep beams do not respond in pure flexure.

Conclusion

More automated and sophisticated methods for calculating blast loadings on structures are being developed, and more sophisticated methods for the design of flexural members and structural motions are becoming available to designers.

Each level of complexity has advantages and disadvantages, and the engineer must be practical in his choice. The engineer must evaluate the level of accuracy required by the structure complexity and analysis cost, and he must then choose the appropriate analysis procedure.

The simple SDOF procedures are adequate for most designs, and complex soil-island analyses are justified in rare cases only. However, as advanced computing power becomes more available to the designer, decoupled finite element analyses will be used more and more.

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In-Structure Shock and Shock Isolation

by

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Abstract

In-structure shock and shock isolation are of interest to both the protective and the seismic design communities. This paper approaches in-structure shock and shock isolation from the protective design point of view; i.e., structures designed to resist the effects of conventional weapons. A major concern in these structures is to protect the mission-critical equipment inside from the shock environment induced by weapon detonations. The design procedure required includes calculating the weapon induced loads, modeling of the structure, calculating the in-structure shock response of the structure, calculating the predicted shock environment in the form of a shock response spectra (SRS), and comparing the predicted shock environment to equipment shock data. The implementation of the design procedure is rarely clear cut and often involves the application of engineering judgement and a clear understanding of the problem.

Introduction

The procedures for computing the in-structure shock response in the form of a shock response spectra (SRS) and the design of shock isolation requirements are discussed. The paper focuses on below-grade structures designed to resist the effects of conventional weapons. In these structures, the goal is to ensure that mission critical equipment is not damaged by the shock induced by conventional weapon ground shock.

Terminology

Before discussing analysis and design methodology, it is useful to define some of the terms that are used.

Ground shock

The term ground shock refers to the particle velocity versus time and pressure versus time waveforms in the soil (ground). For below-grade weapons detonation, the peak velocity and pressure are near the leading edge of the wave, and both velocity and pressure decay with time. As the range or distance from the detonation increases, the peak waveform values decrease and the duration of the waveform increases. Ground shock that is undisturbed by a structure surface is referred to as "free-field" ground shock.

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Soil-structure interaction (SSI)

As the term implies, soil-structure interaction refers to the interaction of the structure and soil during ground shock loading. During this interaction the structure is loaded by the pressure wave but also translates away from the load. This results in a contact pressure that is less intense than the ground shock. This phenomena leads to the conclusion that flexible walls shed ground shock load and, conversely, that stiff walls attract ground shock load.

In-structure shock

In-structure shock refers to the motion of the structure caused by the ground shock loading. This motion can be reported as displacement versus time, velocity versus time, or acceleration versus time. However, acceleration versus time is most often used for in-structure shock calculations.

In-structure shock response spectra

In-structure shock response spectra is the locus of absolute peak response experienced by an elastic single-degree-of-freedom (SDOF) system, excited by the in-structure shock. The SDOF model is a function of frequency and damping.

Equipment fragility

Equipment fragility is the shock that a particular piece or type of equipment experiences when damage occurs. This type of data is very useful because it allows a direct comparison between equipment damage and in-structure shock response spectra. Unfortunately, this type of data is scarce because of the expense of testing equipment to failure.

Equipment tolerance

Equipment tolerance is the shock that a particular piece or type of equipment has experienced in a test without damage. Note that because the equipment is not tested to failure, the failure level is unknown. While this type of data is valuable, it is less useful than equipment fragility data.

Shock isolation systems

A shock isolation system consists of the equipment mass and the support system that isolates equipment from structure motion. The support system usually consists of a spring arrangement between the equipment and the structure. The most often used configurations are base-mounted systems and suspended pendulum systems.

These definitions are the most commonly used but are not universal; e.g., some engineers prefer to use the term "structure media interaction" (SMI) in lieu of "soil-structure interaction."

Modeling In-Structure Shock

The methods available for calculating in-structure shock are free-field modification found in TM 5-855-1 (Headquarters, Department of the Army, 1986), decoupled, and fully coupled analyses. Each of these methods and models are discussed.

Free-field modification method

A simple method for calculating the in-structure shock of buried structures is found in TM 5-855-1. This method accounts for the attenuation of the ground shock and is based on the average free-field acceleration, velocity, and displacement. The response of the model is limited to the principal direction defined by the center of the structure and the charge location. This method is appropriate in cases where local flexural motions can be ignored and has been found to compare well with experimental values.

Decoupled rigid body models

A rigid body model is a model in which the structure is idealized as a lump mass or a series of lump masses. Several rigid body

models have been formulated and analyzed for accuracy and agreement with empirical data. Models that include the effects of soil structure interaction are discussed in papers by Hall (1987), Whitehouse (1988), Weidlinger and Hinman (1988), and Drake, Walter, and Slawson (1989). The rigid body model consisting of a two-degree-of-freedom system that accounts for front wall flexibility, rigid body response, and soil structure interaction is widely accepted.

The advantages of the rigid body model are a simplified formulation, reduced modeling effort, and reduced computation time. As long as the structure is rigid in the direction of the blast, a rigid body model will give reasonable results in that principal direction. An example of a rigid body model is given in Figure 1. In this figure the SSI components are shown acting on the flexible wall elements and the resulting load on the rigid body.

Decoupled discrete models

A discrete structure model is a model in which the structure is idealized by individual elements joined at nodal points. These models usually consist of either a two-dimensional structure model built up from beam elements or a two- or three-dimensional finite element model. In decoupled models the soil is not molded by elements. Instead, a soil-structure interaction algorithm is used to transmit the free-field ground shock to the structure.

The advantage of discrete models is that both global (rigid body) and local (usually flexural) motions can be accounted for and motions can be calculated at several points inside the structure. Thus, flexible structures are much more accurately modeled. Decoupling the soil and omitting the soil finite element grid reduces model formulation complexity and analysis time. In the cases where the structure model is loaded by conventional weapon induced ground shock, nonlinear behavior is usually present only in elements very close to the detonation. Thus, computer programs that allow preselection of nonlinear elements can be used to great advantage.

When only structure motions are required, a fairly coarse finite element grid can often be used without appreciable loss in accuracy because displacement continuity is maintained across the nodal points. The coarse grid approach has the advantage of reducing program execution times. It must be recognized, however, that coarse grid models depart from

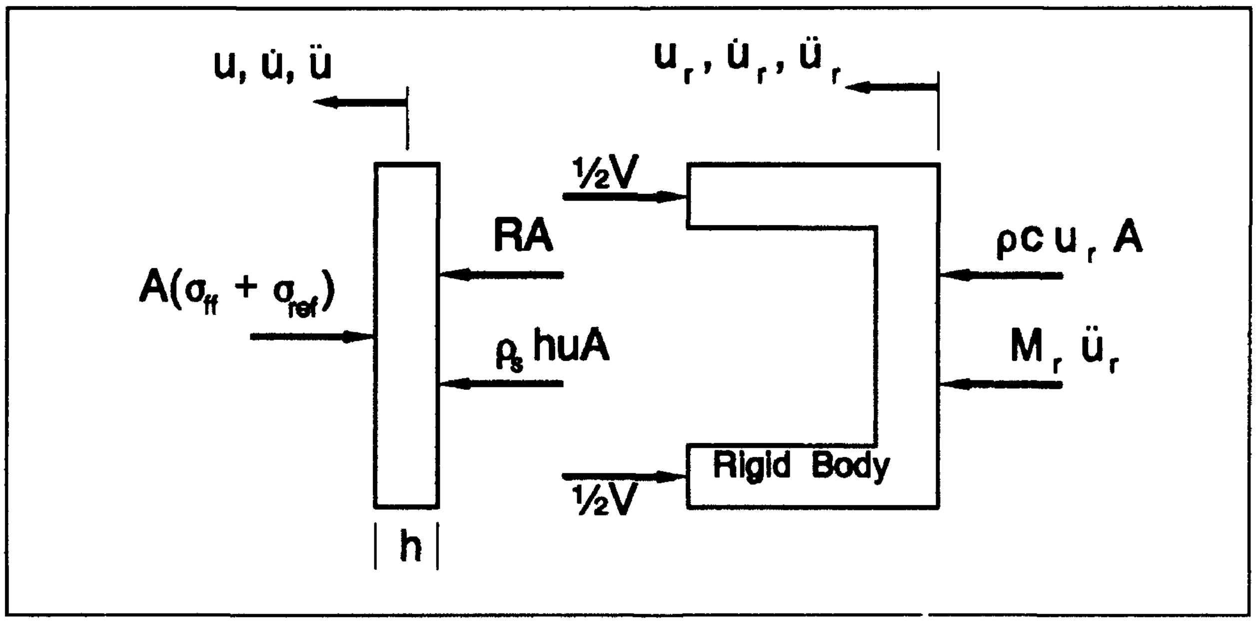

Figure 1. Two-degree-of-freedom model with soil-structure interaction

representing the structure as a continuum, and thus, some behavior details are lost.

An example of a decoupled, three-dimensional finite element model is shown in Figure 2. This model was built up using plate elements to simulate floor, wall, and roof slabs and beam elements to simulate interior columns. The model is shown as a cut-away view with the nodal displacements exaggerated and demonstrates the importance of local response in flexible structures.

Soil island models

Soil island models differ from decoupled models in that the soil media is simulated using finite elements. Only a portion of the surrounding soil can be modeled, thus, the name "soil island." These models are usually loaded by free-field ground shock velocities input at the model boundaries.

In conventional weapons applications, the relative motion between the soil and the structure is very small and is usually ignored; i.e., nodal slip or detachment at the soil-structure interface is generally not modeled. Instead the interaction of the soil and structure is accounted for through a suitable soil material model that limits tension response in the soil material.

When properly formulated, soil island models probably provide the best available calculation of in-structure shock because the true behavior of the ground shock engulfing the structure is captured. The disadvantages of these models lie in the model complexity and longer execution times. A further difficulty is encountered in obtaining credible free-field velocities to load the model. This is usually done through a separate finite difference (or hydro code) source calculation or by a physical experiment.

Shock Spectra

Once the in-structure shock has been calculated at a specific location in the building, a shock isolation analysis for equipment can proceed. The shock isolation analysis of equipment requires the calculation of the SRS for a given in-structure shock. The calculated SRS is an analysis tool used for determining equipment survival in a shock environment.

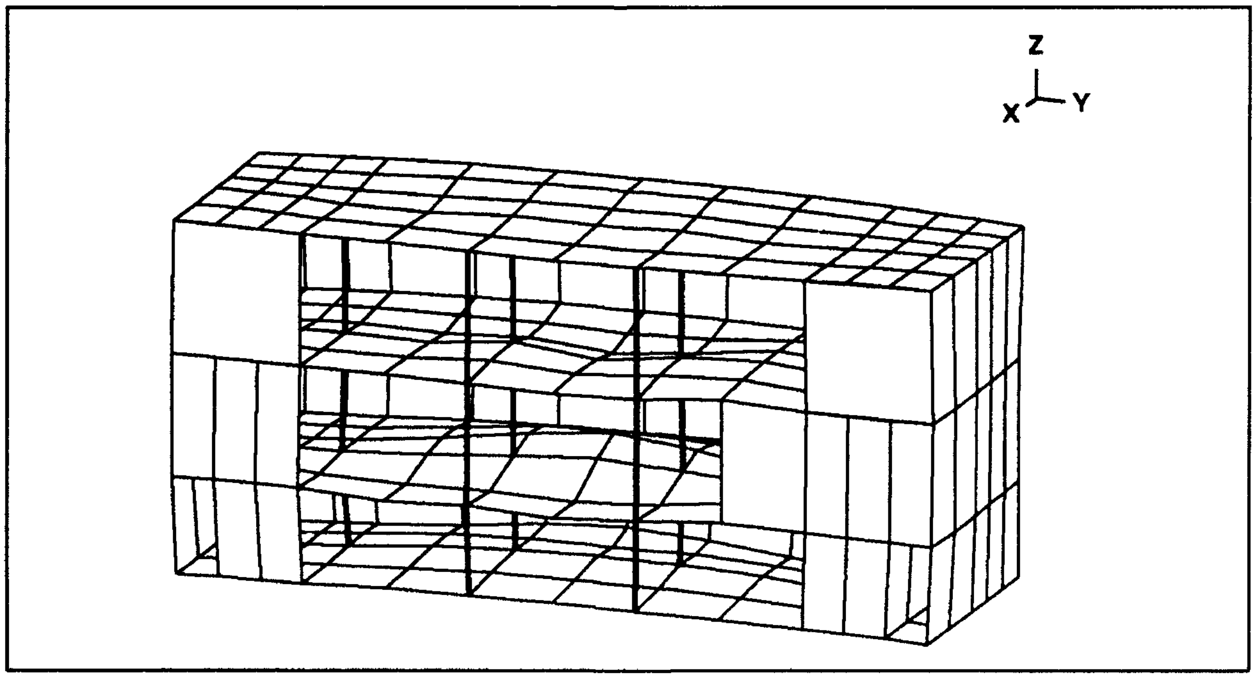

Figure 2. Decoupled discrete element model

It is the comparison of the predicted SRS with the equipment shock tolerance or equipment fragility data that determines if shock isolation is required.

Shock response spectra

Shock response spectra are usually reported on four-coordinate log paper known as tripatite paper (Figure 3). As used in shock isolation design, shock response spectra at a particular location in a structure are a locus of points of the peak relative displacements and accelerations of an elastic SDOF system. Such a system is shown in Figure 4.

A shock response spectra calculation for a particular location is started by selecting a convenient system frequency (mass and spring constant), exciting the system with the in-structure shock, and then calculating the peak relative displacement and acceleration of the system mass. This process is repeated until a locus of frequency versus peak relative displacement and peak acceleration points is obtained. The results for a single calculation are shown in Figure 5.

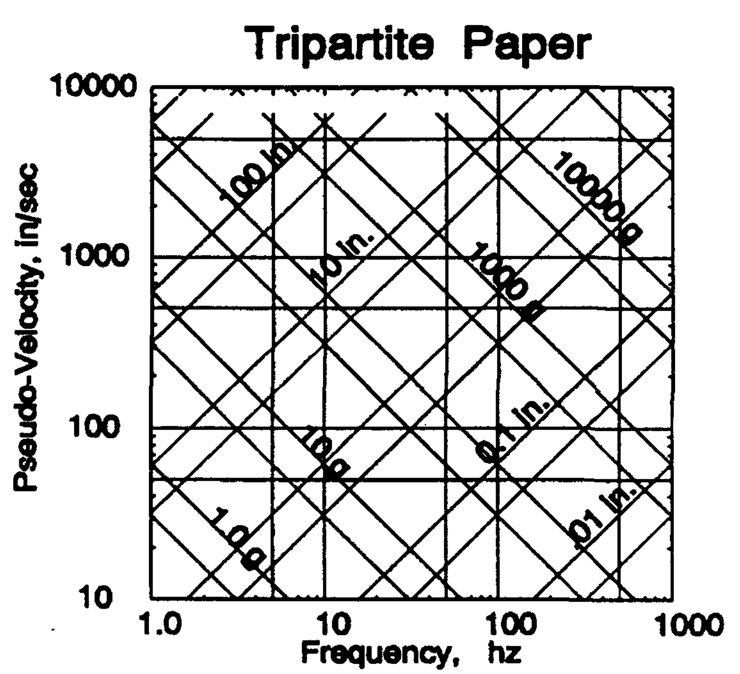

Figure 3. Tripartite or four-coordinate log paper

Shock response spectra or locus of all peak relative displacements and peak accelerations are assumed to be representative of the peak motions that a piece of equipment would experience due to the in-structure shock.

Tripartite plot

Tripartite graph paper is a four-coordinate log paper used in plotting SRS. The four coordinates are for relative displacement, absolute

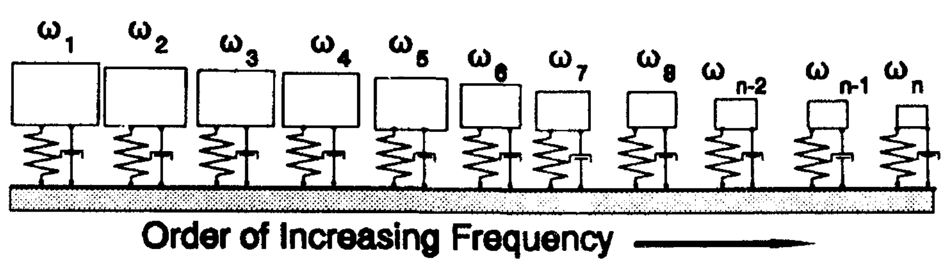

Figure 4. Shock response spectra model

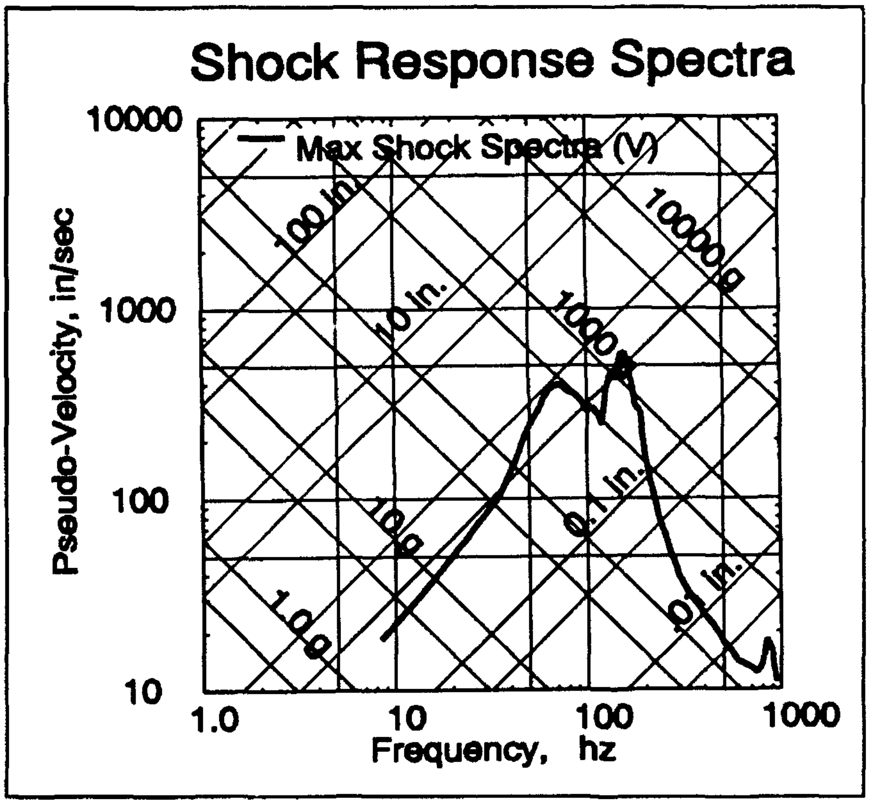

Figure 5. Shock response spectra

ing of 5 to 10 percent of critical are 1.2 for relative displacement, 1.5 for pseudo-velocity, and 2.0 for acceleration. Shock response spectra developed in this manner consist of an envelope of three straight lines as shown in Figure 6.

This method of developing a shock response spectra is generally used in cases where the multiple calculations required to develop the true shock response spectra are not justified. Thus, the amplification method is often used in conjunction with the free-field modification method for determining in-structure shock.

acceleration, psuedo-velocity, and frequency. The locus of peak relative displacements and peak absolute accelerations can be simultaneously shown on a single plot and that system velocity can also be added. Unfortunately, the true peak velocities cannot be plotted simultaneously with peak relative displacement and peak acceleration and harmonic or so called pseudo-velocities are used. A typical tripartite plot of a shock response spectra is shown in Figure 3.

Amplification method

The calculations of many shock response spectra have shown that the locus of peak relative displacements, peak pseudo-velocity, and peak absolute accelerations often move or are less constant over certain ranges of frequency, and that the shock response spectra can be estimated by amplification of the peak in-structure shock. The amplification factors often used in conventional weapons applications for damp-

Computed shock response spectra

Computed SRS are arrived at by calculation of the response history of an SDOF system for all frequencies within a frequency range (usually 1 to 1,000 Hertz). From each of the response histories, the absolute maximum displacement is determined for each frequency. It is this locus of absolute maximum response points that then defines the upper bound of the computed SRS.

Shock Isolation

The determination of the shock isolation requirements is based upon the comparison of the predicted shock response spectra and the equipment shock data. The peak absolute value of relative displacement and absolute acceration are determined for each response history and locus of peak responses versus frequency points form the SRS.

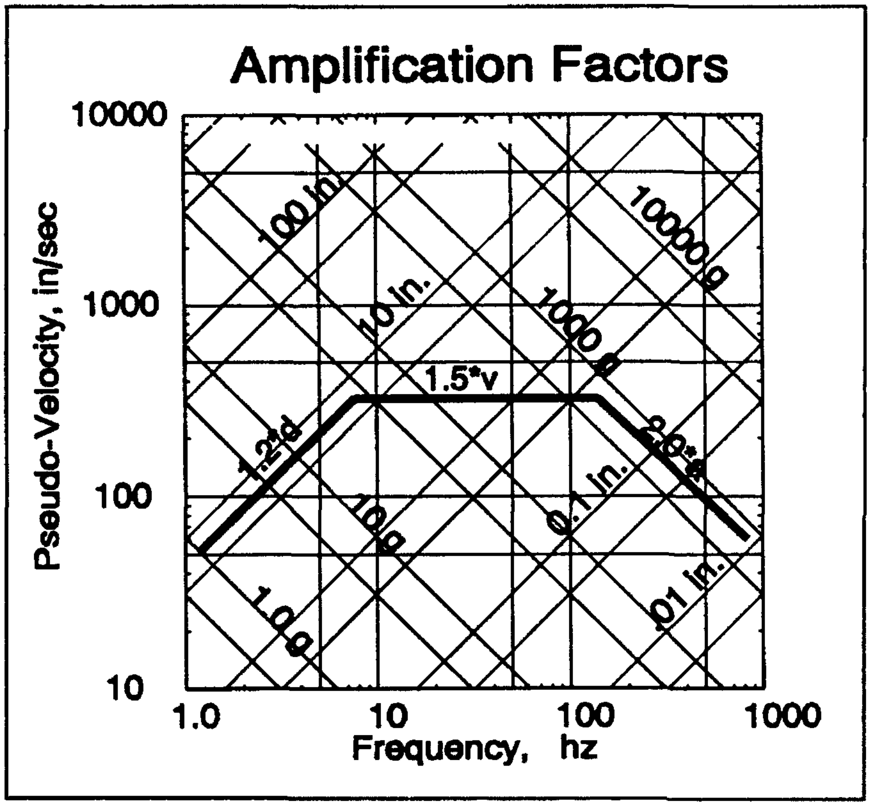

Figure 6. Free-field shock response spectra amplification factors

Predicted shock response spectra and equipment data comparison

The requirement for shock isolation is determined by comparing the predicted shock response spectra to equipment shock data. Typical comparison plots are shown in Figure 7. In this figure the shock data for Equipment A exceed the shock environment for all frequencies and so this equipment does not need to be shock isolated. On the other hand, the shock data for Equipment B exceeds the shock environment at the higher frequency range, and this equipment requires shock isolation. The frequency of the shock isolation is determined by the point at which the equipment shock data and the shock environment lines cross each other. From this comparison, an upper bound of survivability can be established for the equipment.

Equipment Shock Data

The equipment shock data denote the ability of a piece of equipment to survive in-structure shock. Two sources of this equipment data are equipment shock tolerance and equipment fragility.

The source of the equipment shock tolerance data is TM 5-855-1. These data were obtained in a series of tests under the SAFEGUARD program, involving the testing 300 pieces of off-the-shelf facility equipment.

The source of the equipment fragility data is from series of Navy ship-shock qualification trials for two categories of mission critical equipment. This data was then reduced into fragility curves that are functions of damping and probability of failure.

Contractor designed shock isolation systems

When the shock isolation system is relatively simple and straight forward, it can be obtained through a performance specification process. Examples of simple systems include cases in which equipment items are individually isolated and when small equipment groups are placed on a single rigid platform. Performance items to be covered in the construction contract are summarized in Table 1.

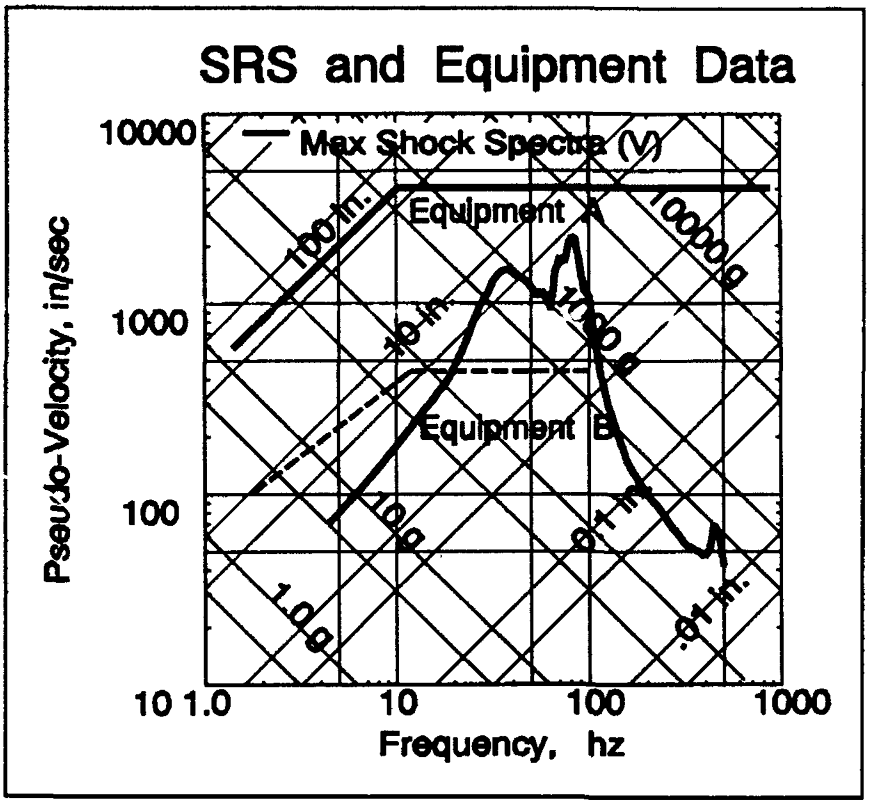

Figure 7. Comparison of shock response spectra and equipment shock data

Table 1 **Construction Contract Requirements** **for Contractor Designed Shock Isolation Systems**	
Performance Item	**Reason for Including**
Isolation Frequency (horizontal and vertical)	So that the spring constants can be determined.
Peak Relative Displacements (horizontal and vertical)	So that the isolator stroke limits are known.
Peak Relative Acceleration (horizontal and vertical)	So that the isolator anchorage can be checked for adequacy.
Type of Isoloation Hardware; i.e., Elastomeric Pad, Helical Spring, etc.	Maintenance, corrosion control, fuel resistance, operating temperature.
Isolation System Configuration; i.e., Number and location of isolators	Ease of bidding. Alternate approved contractor designed configurations should be allowed.

Special shock isolation systems

When the shock isolation system is relatively complex, it should be designed and detailed by the design agency. Examples of complex systems include shock isolated computer floors, large groupings of critical equipment, cases in which equipment is mounted on flexible framing or racks, and cases in which the isolation system configuration is complex or includes nonlinear behavior. In these cases, the shock environment and equipment shock data comparison plot should be used to obtain a first trial spring constant and the entire shock isolation system should be analyzed for the in-structure shock. A trial-and-error approach is used during which the shock isolation system is refined to achieve the desired isolation. These refinements could include varying the spring constants of individual isolators, shifting isolator locations, adding damping devices, etc.

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Gallipolis Lock and Dam Roller Gate Design

by
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Abstract

Gallipolis Lock and Dam is located at mile 279 of the Ohio River. As part of the rehabilitation of the dam, new roller gates will be installed. Although the basic geometry of the gate will remain the same as the existing gates, the internal bracing and stiffening of the load disks will be modified. Also, all connections will be welded instead of riveted.

This paper will describe the design and analysis of the replacement roller gates at Gallipolis Dam. It will include a description of the finite element model used to analyze the gates as well as the methods used to determine loads and allowable stresses.

Results of the finite element model will be compared to a more traditional analysis of the gate.

Background

This presentation is on the design of roller gates. Perhaps we should begin by defining background a roller gate. Several manufacturers such as Rodney Hunt and Armco, as well as ourselves, have designed vertical lift gates with small wheels attached to the sides and have called these gates roller gates. This presentation is not about this type of gate but is instead about a more grandeur size gate in which the entire gates are one large roller. On each end of this roller a ring gear is attached which bears against an inclined rack on the sides of the pier. On one end of the gate a lifting chain is attached and wrapped around the gate.

To be more specific we will be looking at the new replacement gates for the Gallipolis Dam on the Ohio River. These existing gates were installed in the mid 1930's and have some corrosion problems. They are being replaced as part of the new lock project.

To better understand the problems of design, we should take a few minutes to see how roller gates have been involved in the locks and dams of the Upper Mississippi and Ohio Rivers.

Roller gates were developed at the turn-of-the-century in Germany by Dr. Max Karstanjen, Director of the Maschinenfabrik Augsburg-Nurnberg (MAN). European engineers, particularly those in the Scandinavian countries, adopted the design almost immediately. Two German companies, the Krupp Company and the MAN Company, controlled basic patents for the gate. By 1930, European engineers had been using roller gates in dams extensively for over 25 years. However, only ten such structures had been built in the United States, and these were all located on rivers

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where ensuring navigability was not a design concern. There was opposition to building permanent nonmovable dams on the Upper Mississippi and Ohio Rivers. It was feared that these slack water pools would become stagnate and that the dams would increase flood heights, as they would not be capable of passing large ice flows and debris. Therefore, only wicket dams and some lateral dams with locks had been built, and the rivers had remained unrestrained and unreliable.

The completion of the Panama Canal in 1914 had a devastating impact on the upper Midwest by making shipping from the East to West coast cheaper. Adding insult to injury, the ICC doubled the railroad rates for the upper Midwest in 1922. Although agricultural prices had plummeted throughout the nation, the Midwest fared worse than most. Politically, both Presidents Warren G. Harding and Calvin Coolidge opposed direct aid to farmers. They believed in the free market system where the weak farmers would be eliminated. However, Congress was more sympathetic to the plight of the Midwest and tried to indirectly aid the area by developing the waterway system. Herbert C. Hoover, Secretary of Commerce under both Harding and Coolidge, was a strong supporter of waterway improvements. In a speech in 1926 he drew attention to the fact that it was cheaper to ship goods from New York to San Francisco than from Chicago to San Francisco and that the Midwest was becoming more landlocked. Throughout this period the Corps of Engineers actually opposed the development of the 9-ft channel into the late 1920's.

After Herbert Hoover's election in 1928 and his appointment of a new pro-development Chief of Engineers, along with the continual support of Congress, we began the large task of improving the waterway.

The stage was now set for the building of large dams. But what type of gates would be used? At this time tainter gates could only span about 40 ft. This would have led to dams with closely spaced piers. There was still great concern about being able to pass large flows of ice and debris. Hence, the decision was made to go with roller gates because they were durable and could span long distances. The first strictly navigational dam on the Mississippi River was at the Rock Island Rapids which consisted of eleven 100 ft-long roller gates. It was constructed in the early 1930, and interestingly, it was designed in Germany instead of America.

After testing it for passing ice, they found that the gate had to be raised nearly halfway out of the water. This used a lot of water during periods of low river flows. At this point the Corps became actively involved in changing the design of roller gates. The idea of making the gates submergible was developed. The first gate was 3 ft submergible at Dam No. 4 in the mid 1930's, later 5 ft submergible, and finally 8 ft submergible was developed by the end of the 1930's. There were several thesis written during this time period and even one that used photoelasticity, light passing through a plastic model to understand how the loads were transferred. Unfortunately for roller gates, tainter gates had also been evolving. The use of higher strength steel had increased their length from 40 ft to about 70 ft. They also became submergible.

The 1930's seem to have been the decade of the roller gates. Rarely do we see new roller gates being designed today. The new 110-ft gates at Lock and Dam 26R are tainter gates.

Ongoing Work

So much for the past. Our office was contacted by Nashville District, who was working for Huntington District, to design replacement gates for Gallipolis Dam. The design criterion was that the gate could weigh no more than the existing gate because the same lifting equipment was going to continue to be used. They did not want it to weigh much less because it had to resist buoyancy forces and vibration problems.

In addition, the gate had to be basically the same geometrical size. The big difference was going from a riveted design to an all welded design.

As we had worked on roller gates in the past in our district, we felt somewhat comfortable with the design of these new gates, and there was a sense of euphoria. After examining the design analysis of the 1930's and looking at some of the assumptions that they had to make to analyze these indeterminate structures, the euphoria had left and a mild state of panic was setting in. It was at this dark hour that we decided to use a finite element method of analysis.

We were initially planning on using GTSTRUDL, but after talking with a consultant with Power Computing Company, it was decided to use ANSYS because of the size of the model.

The dam at Gallipolis has a clear distance of 125 ft 6 in. between the faces of the pier. It is 130 ft 2 in. from the centerline of load disk to centerline of load disk. It consist of a 22-ft-diam cylinder with an apron that extends down another 7 ft 6 in. to provide a gate that is capable of holding back 29 ft 6 in. of water (see Figure 1).

The cylinder acts as both a beam and also as a torque tube to carry the load from the water as well as the dead weight. The skin of the cylinder is thinner at the ends and has a maximum thickness of 13/16 in. in the center of the span. The skin plate is stiffened by 27 equally spaced stiffeners around the circumference of the gate, and are braced every 8 ft 1-5/8 in. along the length of the gate. Finally, the load is transferred into the load disk at the end of the gate, which in turn transfers the load to the piers and lifting chain (see Figure 2).

Manual calculations were made to design a new framing system of stiffeners and to check end reactions. As the gates are lifted on only one end, the reactions are not symmetrical and hence we could not use symmetry to analyze half of the gate. Thus, we had to model the entire gate. The gate was broken into plate and beam elements to represent the skin plate and stiffeners, respectively. A total of six different types of finite elements were used in the computer model. The output consisted of many pages of element stresses. Computer plots of stress contours were a great aid in visualizing the flow of stress throughout the gate. All drawings were done on CADD (see Figures 3, 4, and 5).

Looking back at the project, one could not help but notice the vast differences between the work done in the 1930's as compared to today. Entire hand computation versus a computer finite element analysis. Hand drawings versus CADD and all riveted construction to all welded construction. However, one cannot help but be impressed by the ingenuity and creativity of the early designers. Also, an emense amount of work went into detailing all of the rivet spacing and to do the construction, which is relatively easy today.

But the basic concept of a roller gate seems to have past the test of time very well. They have proven to be extremely durable and reliable gates. From a design standpoint, they seem to have rather simple lines yet that are elegant structures to work on. They are fun to design.

Finally, I would be remiss if I did not give credit to Stanley A. Posey of Power Computing Company, who was a great help in giving technical assistance during the design and his generosity in letting us continue to have access to the computer for the month that it took the Corps to renegotiate its contract.

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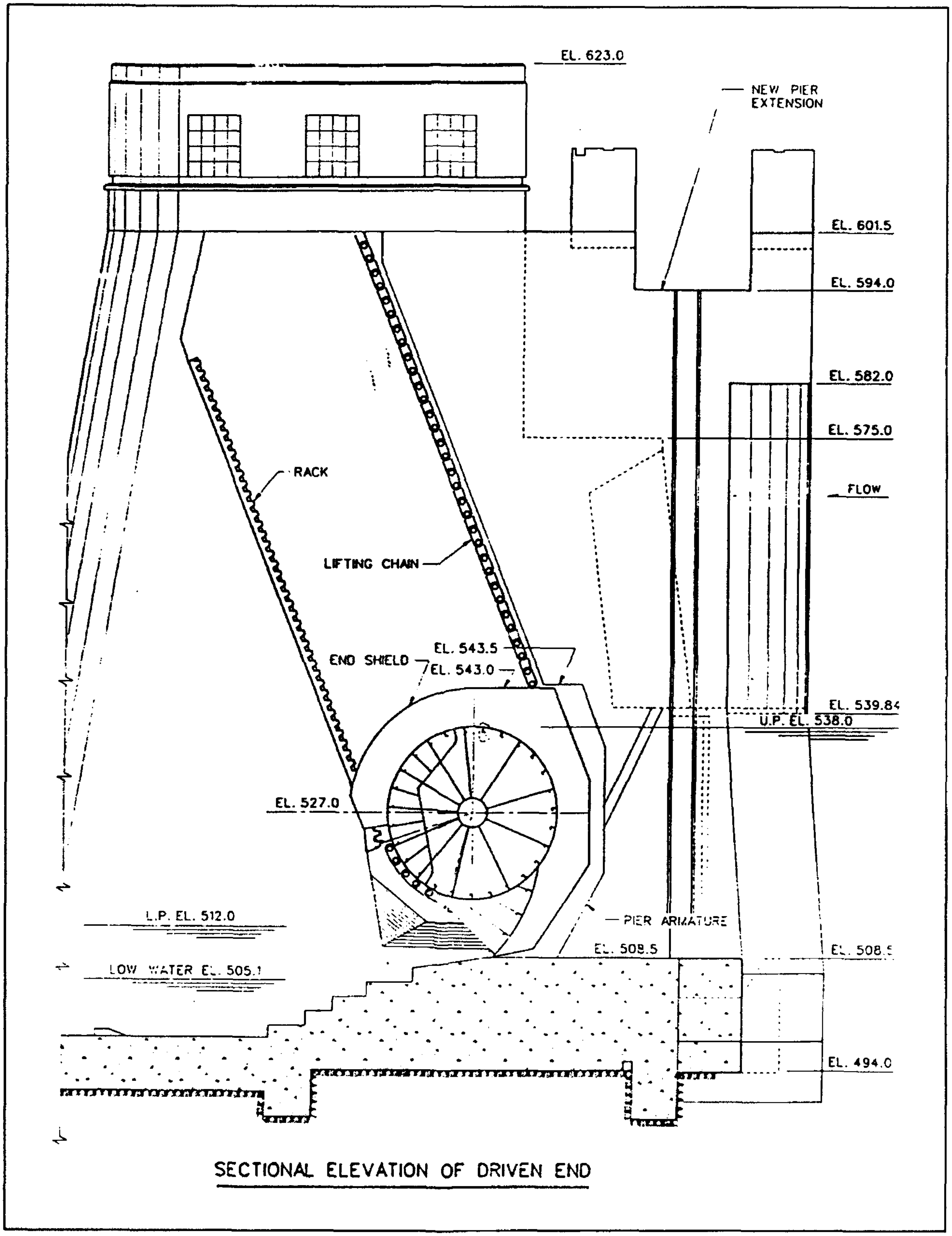

Figure 1. Gallipolis Dam

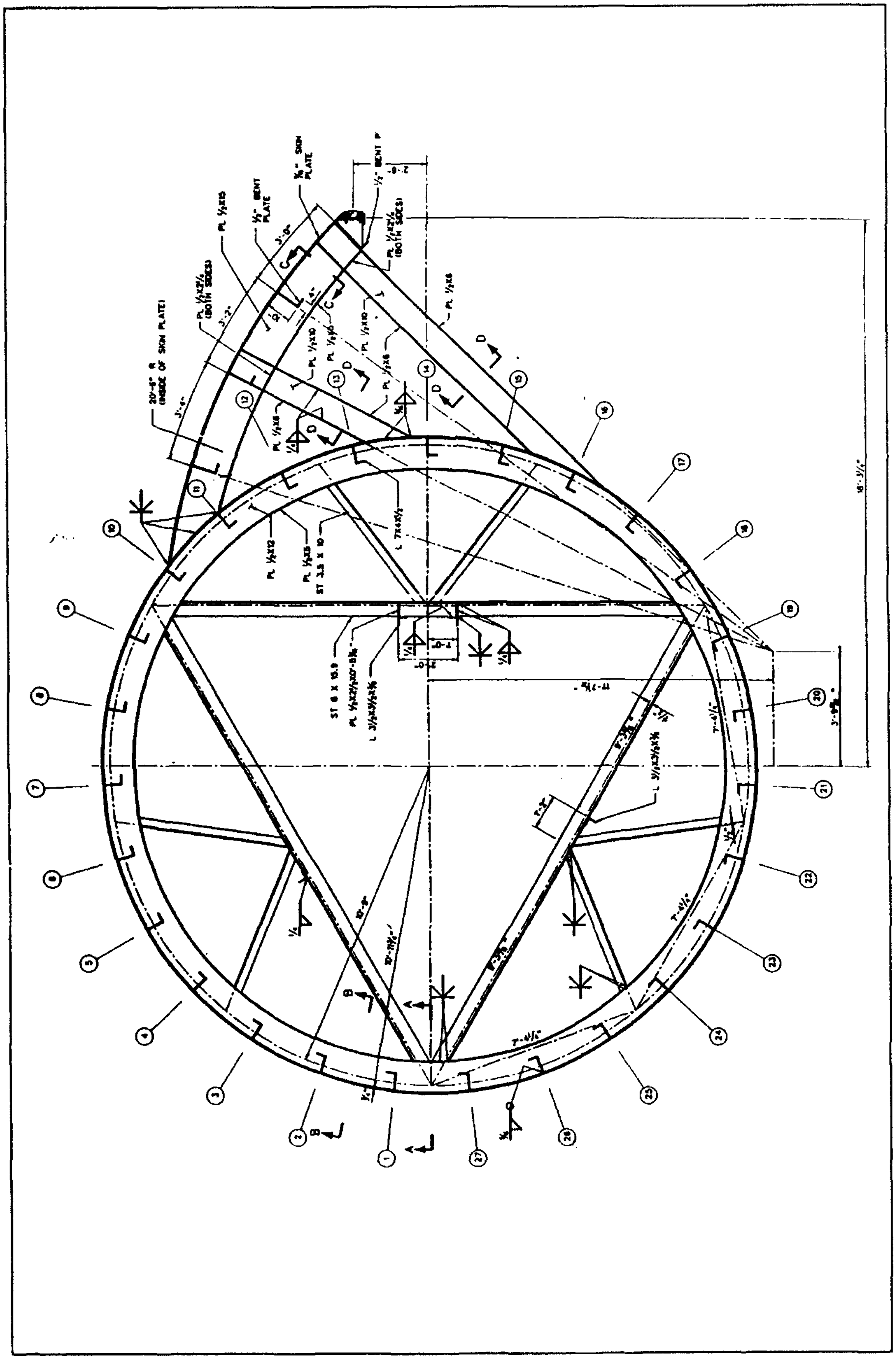

Figure 2. Framing plan

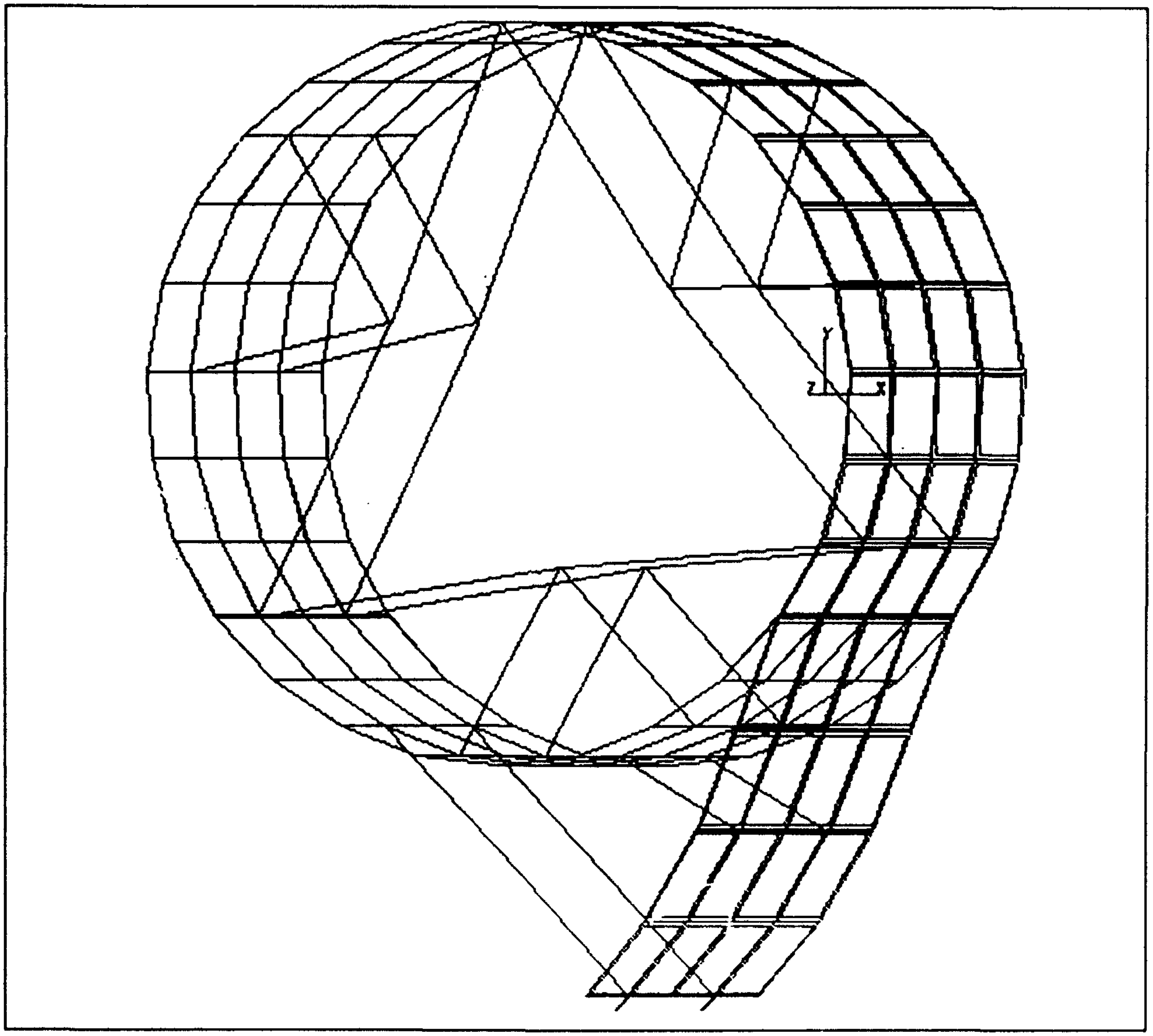

Figure 3. Finite element segment

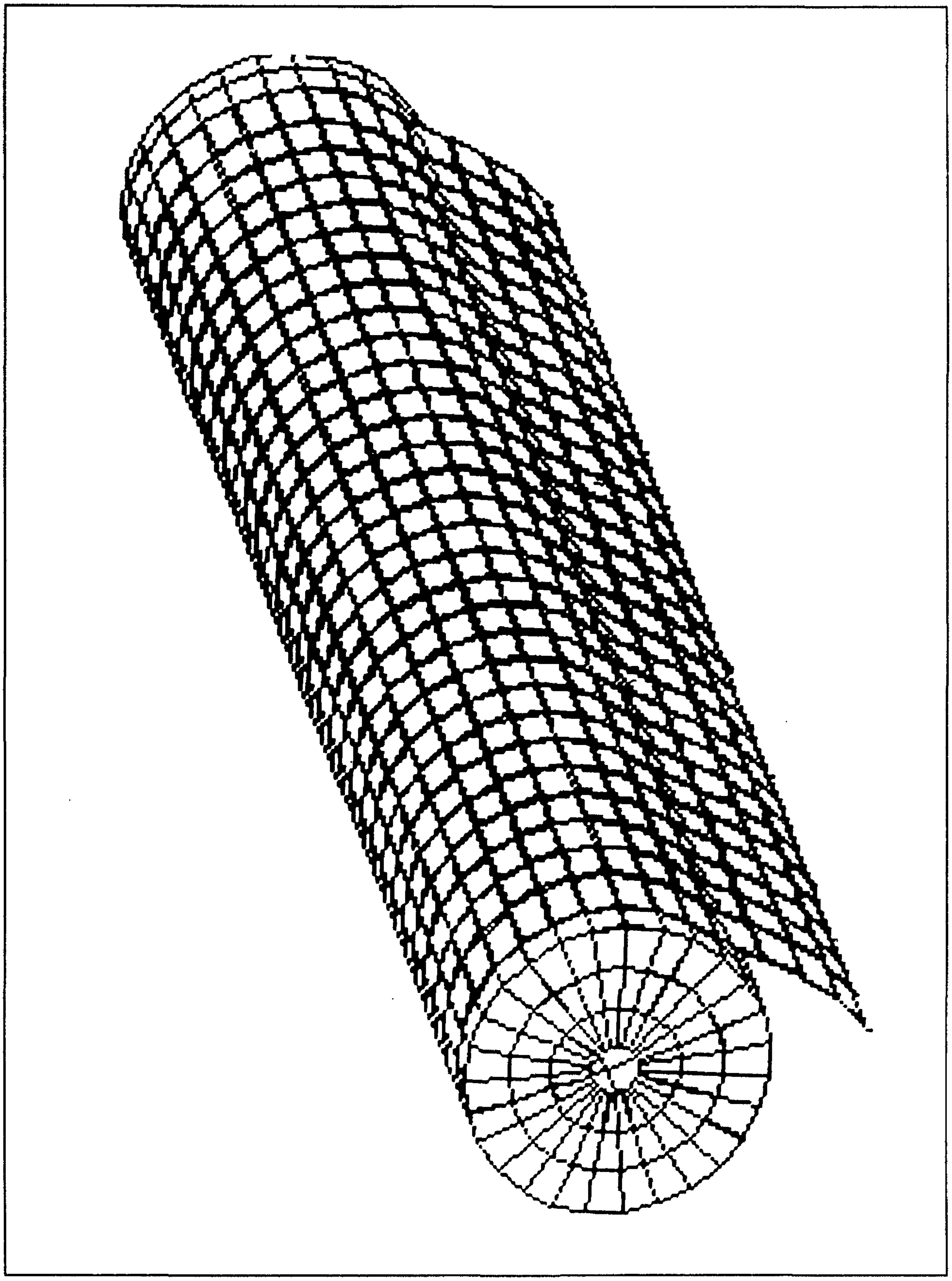

Figure 4. Finite element model

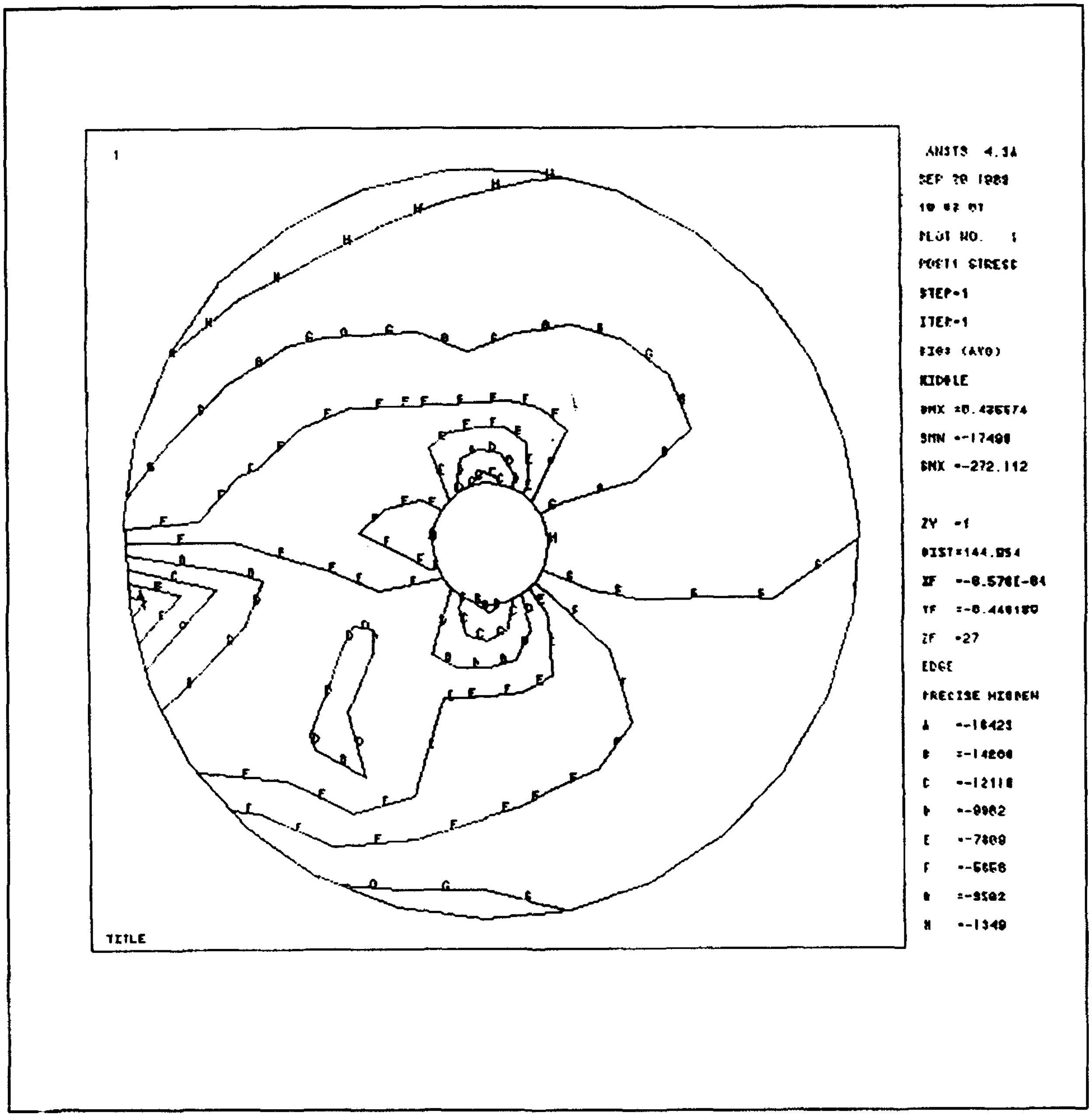

Figure 5. Load disk, stress contours

Determination of Residual Stress and Effects in Thick Section Weldments for Hydraulic Structures

by

Dr. Chon L. Tsai,[1] *John J. Jaeger, PE,*[2] *and Jeffrey S. Sedey, PE*[3]

Abstract

Welding has become the recommended method for fabricating steel hydraulic structures. Current welding codes do not account for interaction between welds placed in close proximity to each other (AWS 1990). During welding of thick plates high tensile residual stress can develop and result in cracking. Cracking recently occurred during fabrication of the tainter gate hitches and trunnion yoke on the Melvin Price Lock and Dam (Bissell 1990). In addition, welding related cracking has also recently been reported on thick sections and plates used for large-span roof trusses (Fisher and Pense 1987, ENR 1988). This paper presents an economical and efficient finite element modeling technique that can be used to numerically analyze residual stress in thick section weldments (Tsai et al. 1989 and 1990) and compares the numerical results with experimental data.

Introduction

For a given welding procedure the temperature history at a point on the weldment is nonlinear and dependent on its distance from the arc. This temperature history is very important, since it determines both the final magnitude and distribution of residual stress that can drive a crack and the ability of the materials to resist cracking locally at the weld. Cracking during fabrication of welded connections has recently occurred on both the tainter gate hitch and the tainter gate trunnion yoke at the new Melvin Price lock and dam. In addition, cracks recently developed on large trusses utilized for the roofing system on the American Airlines hanger in Dallas, TX, and also caused the collapse of the convention center in Orlando, FL.

Once a welding arc is initiated during fusion welding, heat is generated. This heat quickly raises the temperature locally at the arc above the solidus temperature of the base metal, and the material begins to melt. The arc then begins to travel, liquefying material that passes through it. If the welding parameters (voltage, current, and travel speed) remain constant and the material being welded is of uniform thickness, a quasi-stationary thermal condition develops after a few seconds of welding. This quasi-stationary condition ignores the very short time period associated with initiating and terminating the welding arc.

The following discussion proposes a thermal-mechanical finite element method for analyzing the quasi-stationary condition and the

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3 Structural Engineer, US Army Engineer District, Portland; Portland, OR.

interacting effect of welds placed in close proximity to each other. This numerical model provides new insight into the evaluation and determination of welding procedures and processes. Through the use of this numerical model it is possible to investigate the influence weld sequence, joint geometry, and welding parameters have on the magnitude and distribution of residual stress in a welded connection. In addition, it is possible to evaluate the effect of heat on the material properties locally at the weld.

Finite Element Thermal Analyses

The finite element thermal analyses was performed with the general-purpose finite element code ABAQUS (Hibbitt, Karlsson, Sorensen 1984). Since plastic deformation has a negligible effect on the thermal analysis, an uncoupled thermal-mechanical analysis was performed. This implies that the thermal analysis is initially performed and the temperature history from this analysis is used as the loading input for the subsequent stress analysis. Figure 1 illustrates the numerical analysis procedure in flow chart form.

The thermal and stress response of thick weldments is a three-dimensional phenomena. This study transformed the three-dimensional problem into a two-dimensional environment through the use of a ramp heat input model. This ramp heat input model provided a technique to simplify the problem into a two-dimensional slice perpendicular to the direction of welding. To further improve the efficiency of the model, individual weld passes were selectively lumped together. The efficiency of the model was also improved by considering symmetry about the center of the weld. Specific model details are presented later in the paper. The time required to develop the model and the computation time and costs are significantly reduced by using the ramp heat model, lumped weld passes, and symmetry about the weld centerline. The thermal analysis considered conduction and convection but ignored radiation. Since radiation is considered to be more significant in the investigation of fluid flow in the molten pool itself, it is considered appropriate to ignore radiation in a residual stress investigation.

The plates modeled in the analysis were 1/2, 1, and 2 in. thick. Weld joint geometry included single beveled, single-V, and double-V groove butt joints. Temperature dependent thermal properties for the American Society for Testing and Materials A36 material were used for all analyses as shown plotted in Figure 2. Latent heat effects at the solidus and liquidus temperature were accounted for as changes in specific heat. Figure 3 shows the weld pass sequence and welding parameters used to compute energy input while Figures 4a through 4e contain the topography for the numerical models. The finite element mesh was developed so that the refinement increased as you approached the fusion zone. Typically, the mesh consisted of 16 elements through the thickness of the plate near the fusion zone and transitioned to a coarser 4-element mesh in the far field base metal regime where the temperature and stress contours are farther apart. Other investigators have recommended that the element size at the weld area be smaller than or equal to the depth of the weld bead (Boyles, Lee, and Kim 1987). The eight-noded, quadratic interpolation, planar heat transfer element DC2D8 was used from the ABAQUS element library.

The fusion zone mesh was stair-stepped to numerically represent the individual weld beads. Elements representing each weld bead were initially removed and activated for each pass to simulate the deposition of the weld beads. A heat flux was applied to these newly activated elements to generate the heat input. The size of the time increments used was dependent on the magnitude of the temperature gradients. The maximum allowable temperature change between time increments was limited to 200 °F. The temperature data for each time increment was saved as input to the stress analysis. An arc efficiency of 85 percent was used for the net heat input to the weldment.

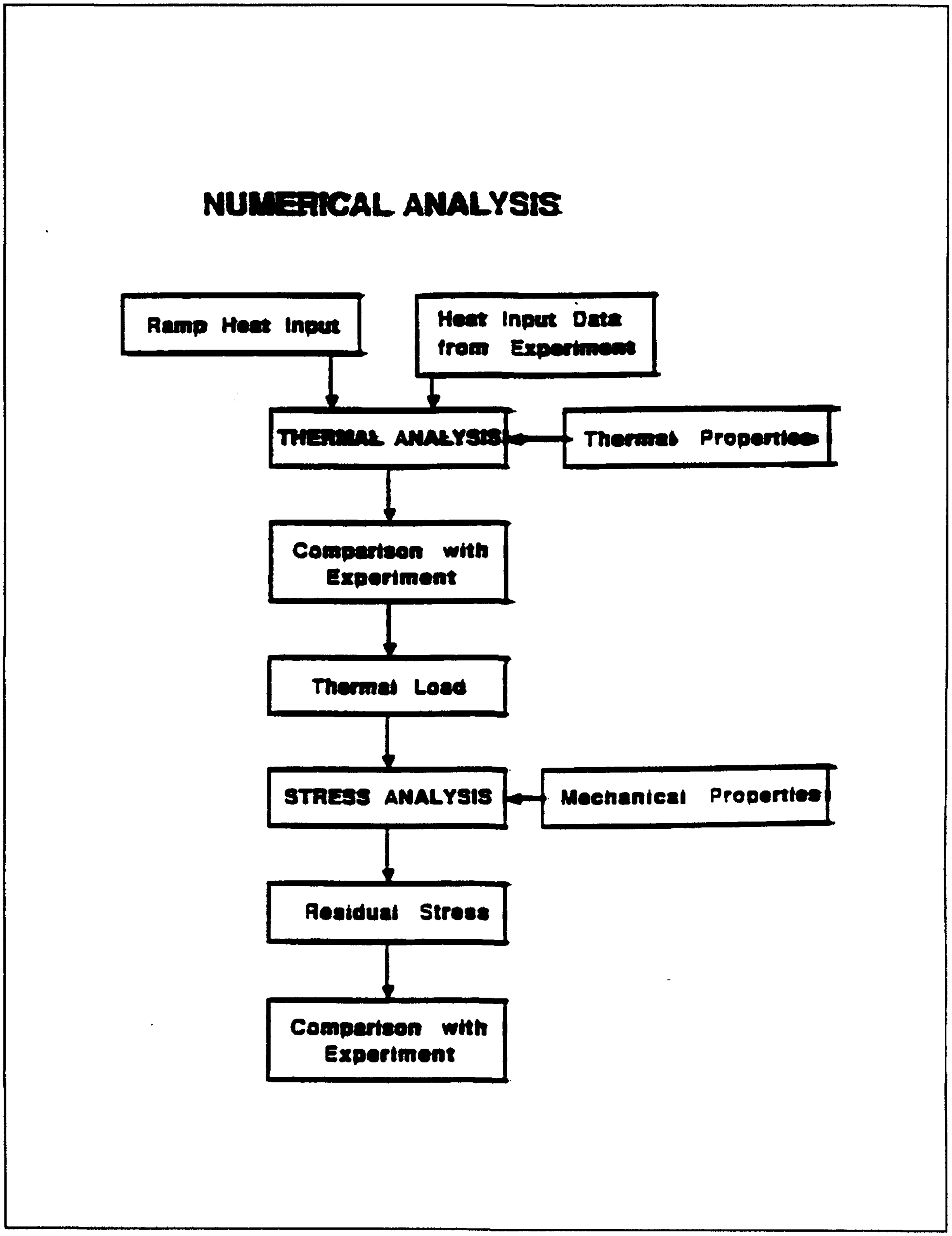


Figure 1. Numerical analysis flow chart

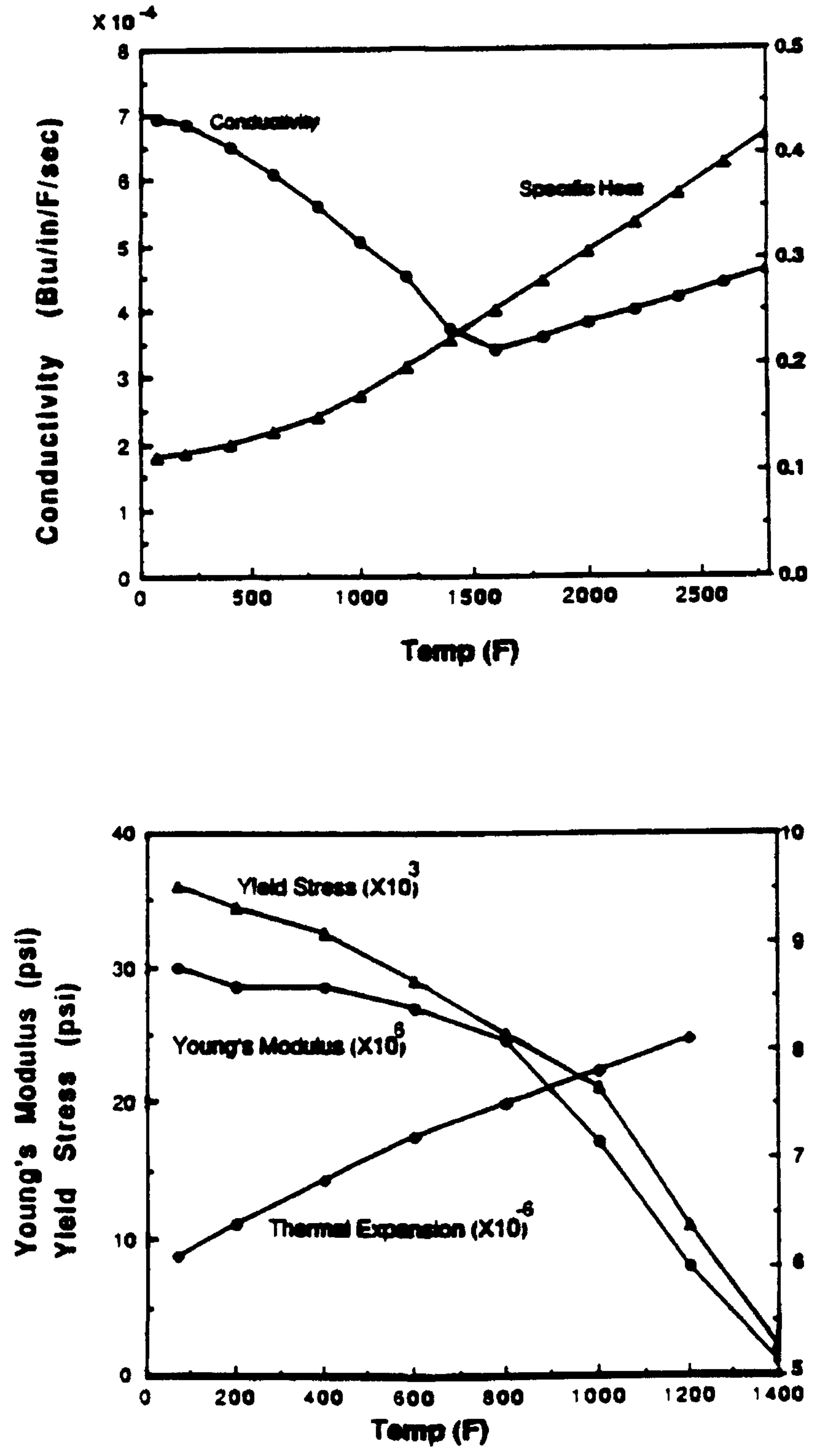
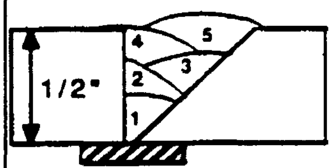
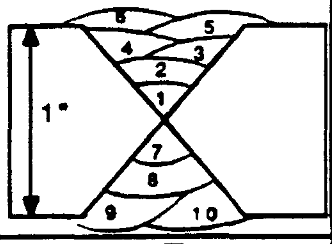
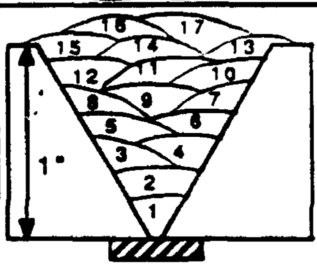
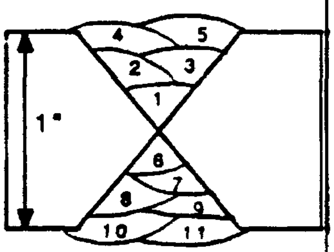
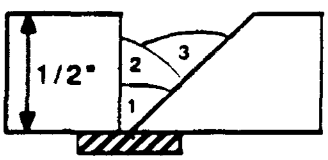
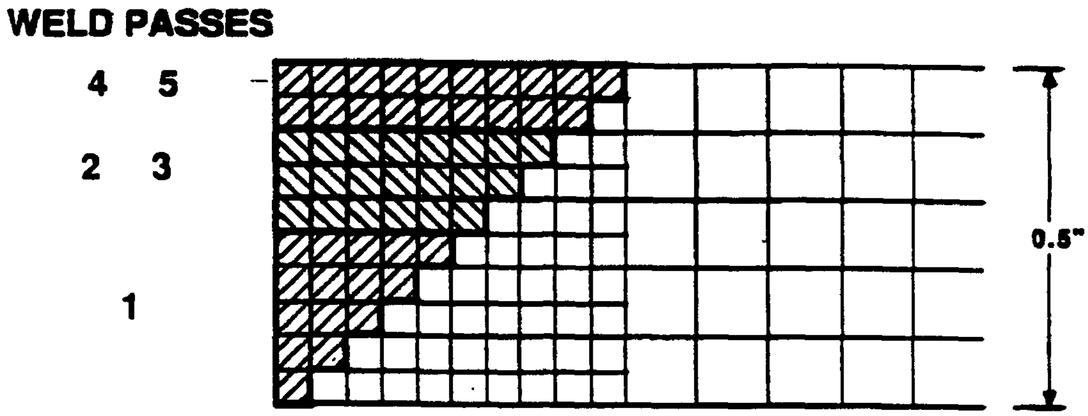


Figure 2. Temperature-dependent material properties

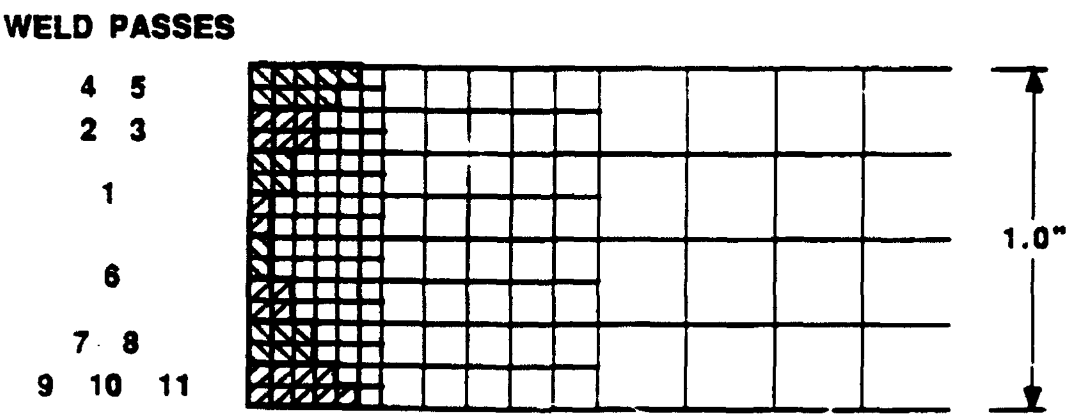
Test No.	Pass Sequence	Pass No.	Welding Parameters Amp (I)	Volt (E)	Speed V(Ipm)	Heat Input (KJ/in)	Remarks
1		1 2 3 4-5	215 205 215 210	24 25 26 24	14.4 14.4 14.4 14.4	18.3 * 18.2 19.8 17.9	
2		1-2 3-5 6 7-10	200 200 200 215	24.2 24.2 24.5 24.5	7.7 12.0 9.9 10.0	32.1 20.6 25.2 26.9	Specimen: Annealed Before Welding
3		1-2 3-6 7-12 13 14-17	195 200 190 195 190	24.5 24.5 24.5 24.5 24.8	6.6 7.9 7.9 7.9 6.6	34.8 31.2 29.7 30.5 34.3	Specimen: Annealed Before Welding
4		1 2-5 6 7-9 10-11	190 215 190 220 250	25 26 25 26 27	7.9 11.1 7.9 11.1 11.1	30.7 25.7 30.7 26.3 29.8	
5		1 2-3	205 215	25 26	11.1 11.1	23.5 25.7	Welding Stopped after 3 Passes

* HEAT INPUT = $\frac{60\ N\ EI}{V}$ WHERE ARC EFFICIENCY (N) = 85% FOR GMAW PROCESS

Figure 3. Pass sequences and welding parameters of each pass

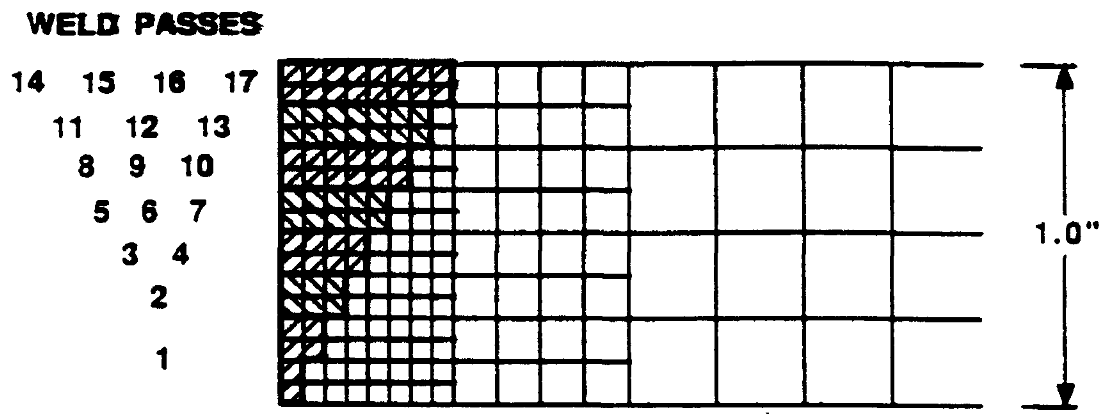


a. Finite element mesh for the 1/2-in.-thick plate

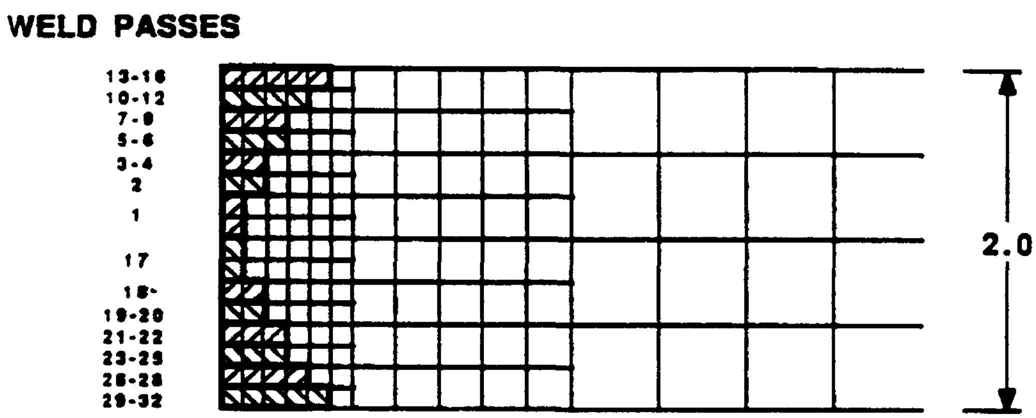


b. Finite element mesh for the 1-in.-thick plate with a double-V groove

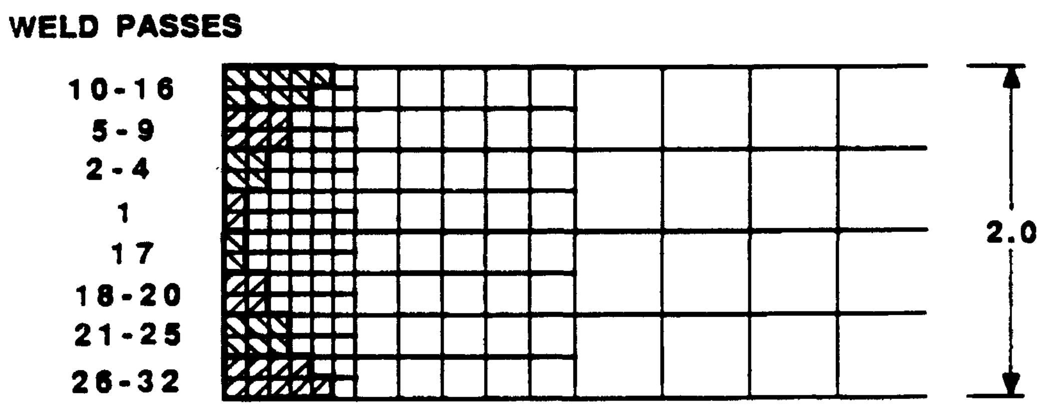
Figure 4. Topography for the numerical models (Continued)



c. Finite element mesh for the 1-in.-thick plate with a single-V groove



d. Finite element mesh for the 2-in.-thick plate (the first lumped pass model)



e. Finite element mesh for the 2-in.-thick plate (the second lumped pass model)

Figure 4. (Concluded)

Heat flux was applied to the model gradually by using a ramp heat input model that was developed to allow variable ramp times. The ramp heat input model was developed to avoid numerical convergence problems due to an instantaneous increase in temperature near the fusion zone and to provide for out of plane modeling of the moving arc in the two-dimensional model. The general amplitude-time function for the ramp input model is shown in Figure 5. The total welding time for the arc to travel across the unit thickness of the finite element model was t1 + t2. Ramp times (t1 and t3) from 10 to 100 percent of the total actual heat input time (t1 + t2) were considered in the study of ramp time effect. The total area under these various ramp heat cycles was kept constant to ensure that the same total heat input to the model was maintained.

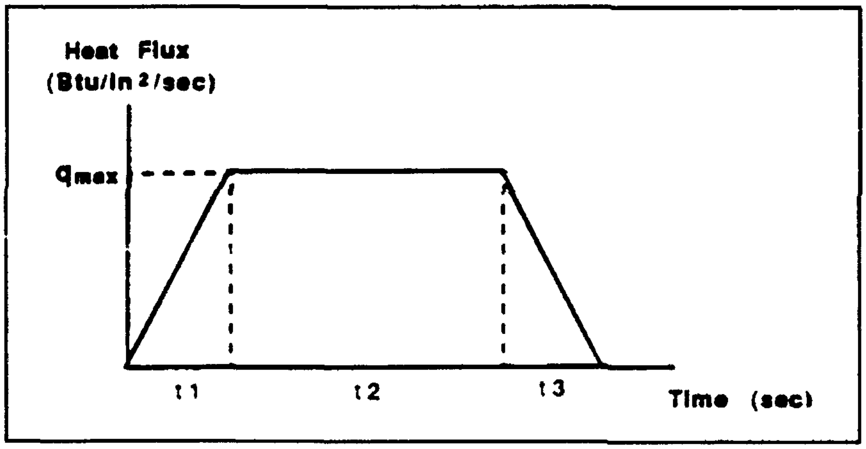


Figure 5. Numerical model of heat flux input

The thermal analysis temperature history results for the 1/2-in. thick plate were compared with experimental data to achieve the best ramp heat input model. Figure 6 shows the temperature profiles at 1/4 in. from the joint edge preparation for the first pass of the 1/2-in. thick plate for the various ramp heat input models. The temperature profiles for large ramp times were shifted to the right indicating more time was required to reach peak temperature. The maximum difference in time to reach peak temperature was approximately 2 sec. The maximum temperature difference during the heating cycle was about 180 °F. The temperature difference during the cooling cycle was very small relative to the heating cycle. Additional numerical and experimental comparisons were made at 1/2 in. from the edge preparation. The comparison showed similar trends with the data 1/4 in. from the edge preparation, but the effect of ramp time was less dominating as the distance from the edge increase. Generally, a ramp time of 20 percent of the total weld time (t1 + t2) gave the best correlation with experimental data. Consequently, this value was used for the temperature analyses of the 1- and 2-in. weldments.

Two modeling techniques were used to input the heat flux in the fusion zone. The first model considered each weld bead individually with the heat flux for each bead being applied along the top surface of the elements representing the bead. The second model grouped individual beads into larger lumped passes. The total heat flux was maintained; however, the computational time is reduced. For the 1-in. thick plate with a double-V groove, 11 actual passes were simulated by 6 lumped passes, 17 actual passes for the single-V groove were simulated by 7 lumped passes, and the 32 actual passes for the 2-in. thick plate with a double-V groove were simulated by 8 passes. The effect of modeling individual passes versus lumping weld passes on the final residual stress distribution is discussed in the finite element mechanical model section.

Finite Element Mechanical Model

The temperature history obtained from the thermal analysis was used as input to the stress analysis to calculate thermal strains and stresses for each time increment. The stress

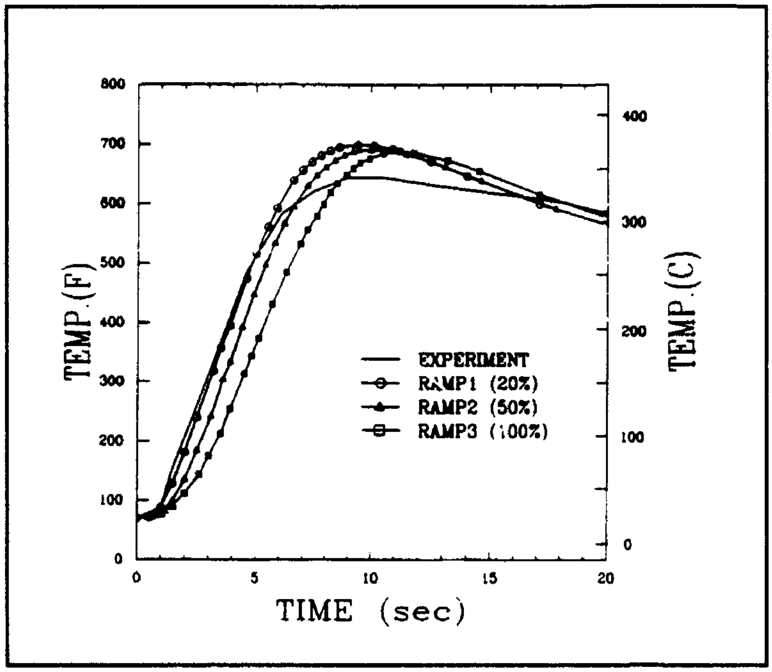


Figure 6. Effect of ramp times for the first pass of the 1/2-in.-thick plate (1/4-in. from the edge preparation at the top)

analysis utilized the same mesh as the thermal analysis. The thermal strains and stresses were accumulated to produce the final state of residual stress. A generalized plane strain analysis was used to analyze a typical cross section perpendicular to the welding direction. The generalized plane strain conditions assume that the model is bounded by two initially parallel planes in the thickness direction. These planes may move as rigid bodies with respect to each other so that strain varies linearly throughout the cross section. The analysis was representative of quasi-stationary conditions.

Similar to the thermal analysis, elements in the fusion zone were incrementally activated to model the weld passes as they were deposited. Free boundary conditions were modeled for the free surfaces except at the centerline of the cross section where symmetry conditions existed. Volume changes due to phase transformations were neglected. Initial stress and strain in the model was assumed to be zero since the experimental plates were stress relieved before welding. The mass of the plate was not included in the analysis.

Individual and lumped weld passes were modeled. The results of these analysis were compared with experimental data. Both models resulted with tensile residual stress near yield at the weld centerline with decreasing tensile stress transitioning to compressive stress outside the weld area. The lumped models however tended to have wider tensile stress fields at the weld with lower longitudinal compressive stress outside the weld area and higher transverse tensile stress at the weld centerline. Generally the experimental data was between the two heat input models with closer correlation to the individual weld pass model. Figures 7 and 8 show representative comparison plots of the longitudinal and transverse stress for the individual and lumped weld pass models.

Conclusions

The magnitude and distribution of residual stress from multipass welds can be evaluated through the use of finite element methods. This study (Tsai et al. 1989) proposes the use of a ramp heat input model with 20 percent of the actual heat input time utilized as the ramping time. The ramp model avoids numerical instability and reduces computational costs by transforming the three-dimensional problem to two dimensions. Computational costs can further be reduced by lumping weld passes, however additional research must be performed to determine the maximum number of individual passes that can be lumped together. This modeling technique has application toward evaluating the interacting effects of critical welds placed in close proximity to each other. The model can be used to determine critical cooling rates and predict final material microstructures in the weldment. Through the use of this model the optimum welding process, procedure, and welding sequence can be determined so that residual stress is minimized and

material properties locally at the weld are improved. The use of this type of thermal mechanical model will improve the quality of Corps critical weldments.

Acknowledgements

This project would not have been possible without the support and guidance of Dr. N. Radhakrishnan, Chief, Information Technology Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, MS. The initial modeling on flat plates was expanded and applied to thick-section rolled shapes with the support of the US Army Engineer District, Portland (Tsai et al. 1990). Mr. Jeffrey Sedey was the primary point of contact at the Portland District for this project. Special thanks is also given to Bethlehem Steel, Beasley Construction Company, The Lincoln Electric Company, and Oregon Iron Works for the labor, equipment, and material donated for the experimental portion of this study.

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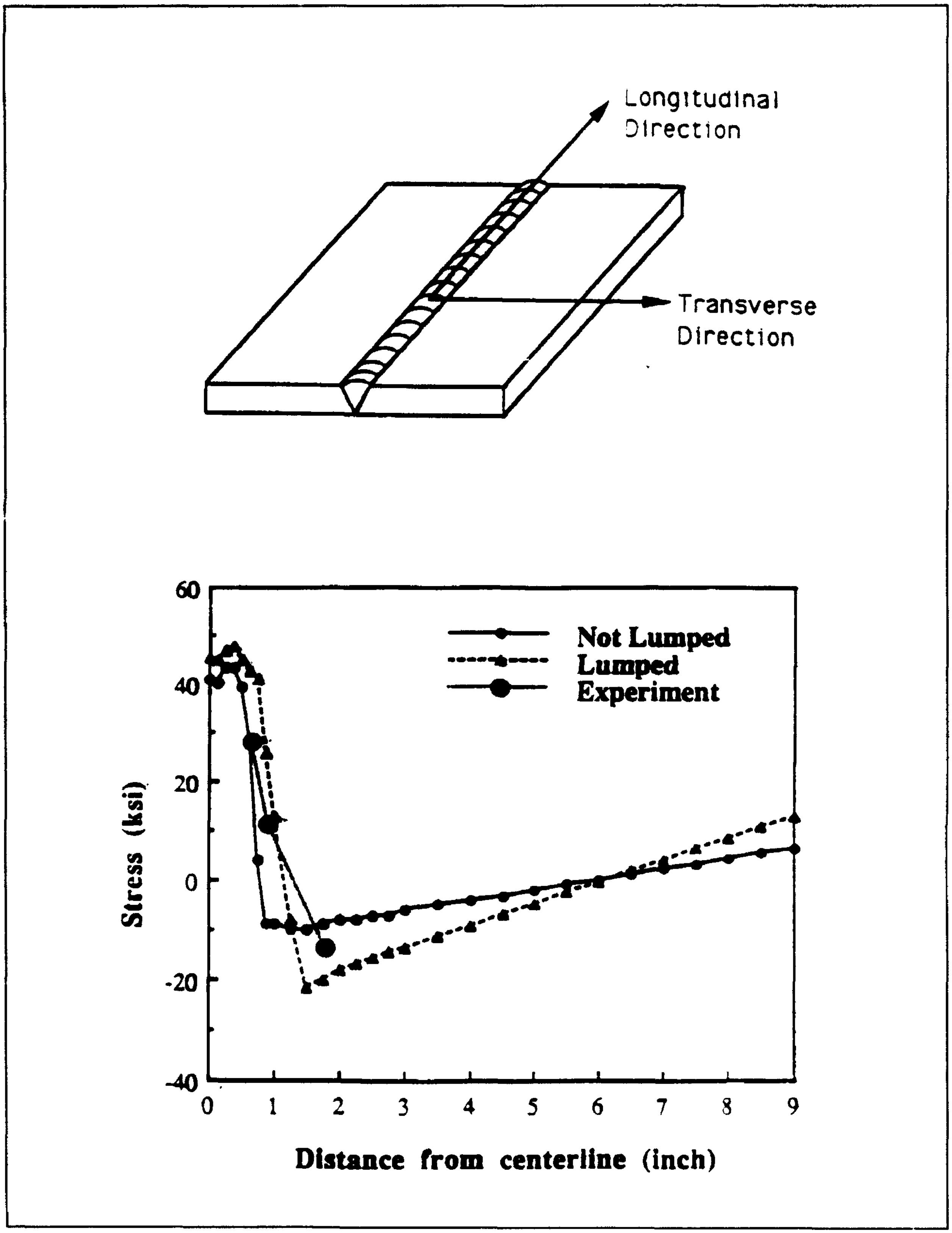

Figure 7. Longitudinal stress at the top surface of the 1-in.-thick plate with a double-V groove

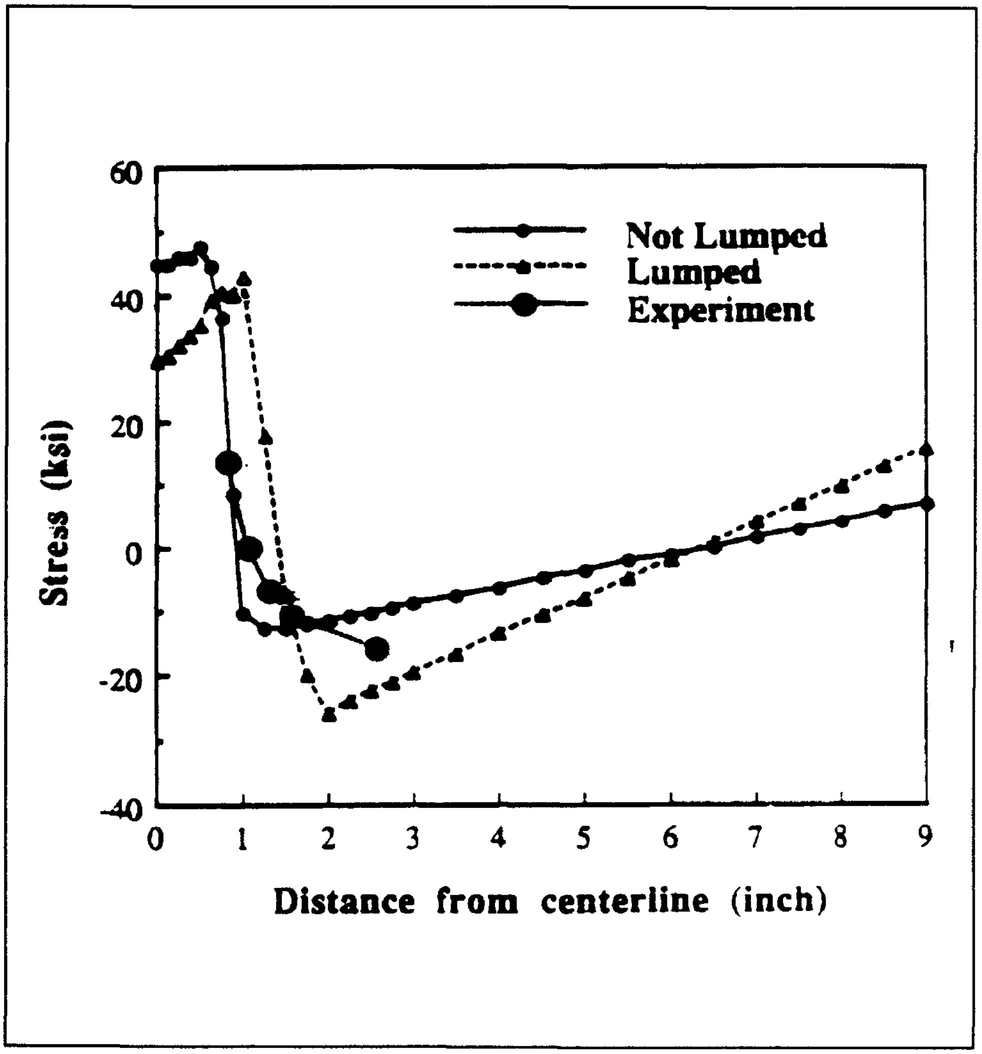

Figure 8. Longitudinal stress at the top surface of the 1-in.-thick plate with a single-V groove

Robert F. Henry Spillway Trash Gates

by
Michael D. Thompson[1]

Abstract

The Robert F. Henry Lock, Dam, and Powerhouse is located on the Alabama River about 15 miles southeast of Selma, AL. The powerhouse has had a trash and debris buildup problem at the intake area since it was constructed in 1975. In 1988, the problem became so severe that the powerhouse came close to shutting down because the raw water, cooling water, and turbine intakes were clogged with debris. In the past, trees, logs, limbs, and other various types of debris have been passed through the spillway gate adjacent to the powerhouse. Because of the erosion potential at the end of the stilling basin apron caused from the energy released from high-flow velocities, debris could be passed only two to three periods a year when the tailwater was high enough to prevent this erosion.

After studying various alternatives of removing debris, the decision was made to construct a trash gate in the upper portion of the existing adjacent spillway tainter gate. Since the majority of the debris was floating, the trash gate could be lowered, passing debris downstream through the trash gate. This could be done on a regular basis preventing a heavy debris buildup that would clog intakes or become saturated and sink.

With the trash gate added, the tainter gate had to be redesigned. The upper 8 ft of the gate skin plate and ribs were removed, hinges added and replaced. Hydraulic cylinders, stiffener ribs, and various other structural steel members were added for the gate to operate properly and be structurally sound. The additional weight added to the tainter gate was critical because the existing gate machinery used to raise and lower the gate was to remain.

By adding this trash gate, the debris problem should be eliminated at Robert F. Henry Powerhouse. With the success of this trash gate system, other projects with debris problems may want to take a serious look at this innovative means of trash removal.

Background

The construction of Robert F. Henry Lock and Dam was authorized by Congress in Section 2 of the River and Harbor Act adopted in March 1945. The project is located on the Alabama River in the central part of Alabama about 15 miles southeast of Selma, AL.

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The project is made up of the following components: a 3,000-ft earth dike connecting the right bank to a four-unit powerhouse; a gated spillway in the river channel consisting of eleven 50-ft- wide by 35-ft-high tainter gates for a spillway length of 646 ft; a nominal 84-ft-wide by 600-ft-long lock; and an earth dike connecting the lock to an earth mound on the left bank. This dam creates a reservoir 80 miles long and has an area of approximately 12,300 acres at normal pool level.

History

Since the powerhouse began operating in 1975, there has been a problem with trash and debris buildup in front of the powerhouse turbine intakes. The buildup became so great at times that turbine efficiency was reduced. The types of trash and debris primarily consisted of trees, logs, limbs, aquatic plants, and weeds, but also included old tires, refrigerators, and various other types of household refuse. On one occasion, the intakes for the raw water and cooling water became clogged and almost caused the plant to shut down. Since there was no onsite trash raking system provided, one method of removing trash and debris was to bring in the snagboat ROS. The trash and debris was removed from the intake area and placed in a barge and pushed to an offloading area. The trash was then loaded from the barge into dump trucks and hauled to a disposal site. Various problems were associated with hauling of the trash and debris. The water that drained from this debris spilled onto the roads and highways, making them a safety hazard. When the material was disposed, it was still saturated, and after a short period of time, the odor became a problem. Over the years, disposal areas became full, and locating new areas was a major concern.

Another method used in removing the trash and debris was by passing it through the gated spillway adjacent to the powerhouse. This method could be accomplished only when the tailwater was at elevation (el) 103.0, 23 ft above normal tailwater, or higher, which occurs only two or three periods a year. The reason for this was the erosion potential at the end of the stilling basin apron caused from the energy released from the high-flow velocities when the gate was in the raised position.

The majority of waterlogged debris could be eliminated if the floating debris could be flushed downstream before it had time to sink in front of the intakes. With this idea, Operations asked Engineering Division to consider what the alternatives were to locate some type of trash gate in the spillway adjacent to the powerhouse.

Alternatives

Several alternatives of trash and debris removal through the spillway were considered. Alternative No. 1 was to remove the tainter gate in the No. 1 spillway slot (adjacent to the powerhouse) and place concrete up to a desired crest elevation (Figure 1). The remaining height, from the new crest elevation to the top of the existing gate elevation, would be used to flush the debris downstream by means of a vertical lift trash gate.

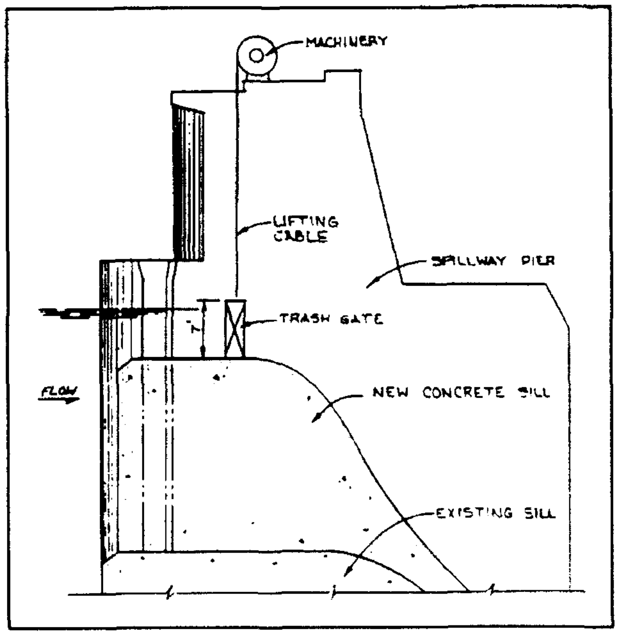

Figure 1. Alternative No. 1

Alternative No. 2 was to replace the existing tainter gate with a three-sectional vertical lift gate (Figure 2). The two bottom gate sections

would each be approximately 14 ft high, and the top gate section would be approximately 7 ft high. Each section would have a separate design to resist the different water loads. A slot about 1-1/2 to 2 ft deep on each pier would be required to guide the gates during raising and lowering for the gates to seal. This could be done by cutting into the existing piers or adding additional concrete to the face of the piers, reducing the spillway width by 3 to 4 ft.

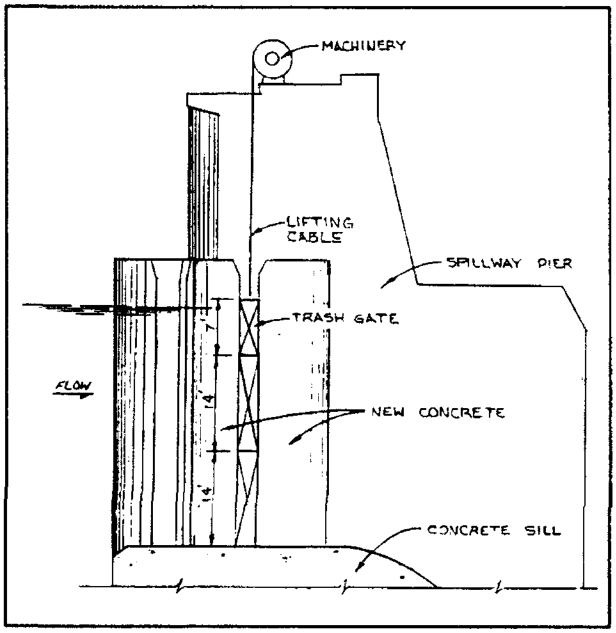

Figure 2. Alternative No. 2

A third method considered was to use the top portion of the tainter gate as a trash gate (Figure 3). This could be accomplished by cutting out the upper section of the gate's skin plate and ribs and reattaching with hinges. With this design, the trash gate could be lowered, and the trash and debris would flow over the top of the tainter gate. Hydraulic cylinders would be used to raise and lower the gate.

There were more drawbacks associated with Alternatives No. 1 and 2 than with Alternative No. 3. Alternatives No. 1 and 2 would reduce the capacity for passing flow during high-water periods. This loss of spillway capacity was not desirable from a hydraulic standpoint. Both alternatives would also require a different lifting system than is used in raising the existing tainter gate. The vertical

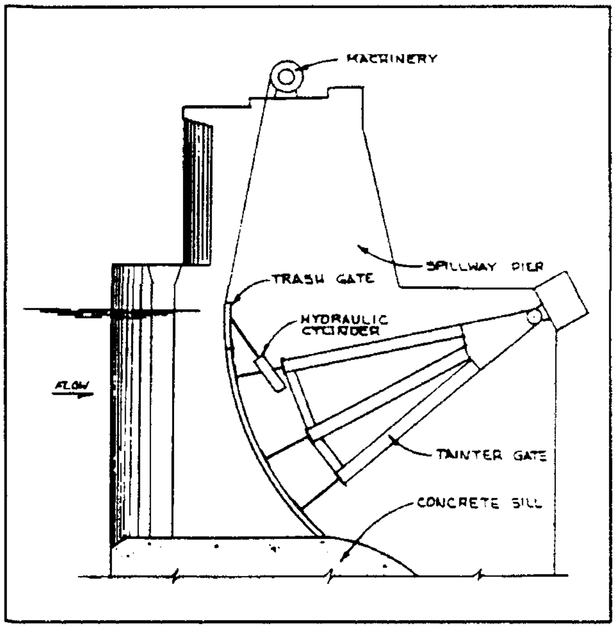

Figure 3. Alternative No. 3

lift gates would have to be fabricated, and extensive concrete work would be necessary. Alternative No. 3 would be more adaptable to the existing situation. The existing lifting system could be used to raise and lower the tainter gate, and only an additional small hydraulic system would be necessary to operate the trash gate. With this alternative, the spillway could still be used to pass flow during high-water periods.

Recommended Plan

After evaluating the proposed alternatives, comparing cost of construction, constructability, ease and cost of maintenance, and cost of operating, Alternate No. 3 was recommended for the trash gate.

The modification of the existing gate consisted of cutting a section approximately 41 ft wide and 8 ft deep out of the top, center portion of the skin plate and ribs. Hinges were designed to transfer the shear load from the cantilevered ribs. Because the ribs were hinged and could not transfer moment, a beam was located near the top of the trash gate leaf to support the ribs. This beam, a W18x106, was designed to support 20 ribs and span approximately 46 ft. Attached to each end of the beam, a hydraulic

locking cylinder is connected to a telescoping locking pin. The locking pins are constructed of 6-in.-diam extra strength pipe. These pins are inserted by the locking cylinders into a pipe sleeve that is mounted at each end of the tainter gate. The sleeves are the supporting members for the beam when the trash gate is in the closed position. Four stop pads are located along the top girder of the tainter gate to support the beam when the trash gate is in the open position. These stop pads consist of a plate steel weldment with a neoprene bearing pad attached to the top plate (Figures 4 and 5).

The trash gate is opened and closed by a hydraulic cylinder located at each end of the trash gate. A hole was cut in the web of the top tainter gate girder at each cylinder location for the operating cylinder to be mounted. The area around the hole had stiffeners added to resist any additional stresses. Each cylinder's push rod was connected to a pin

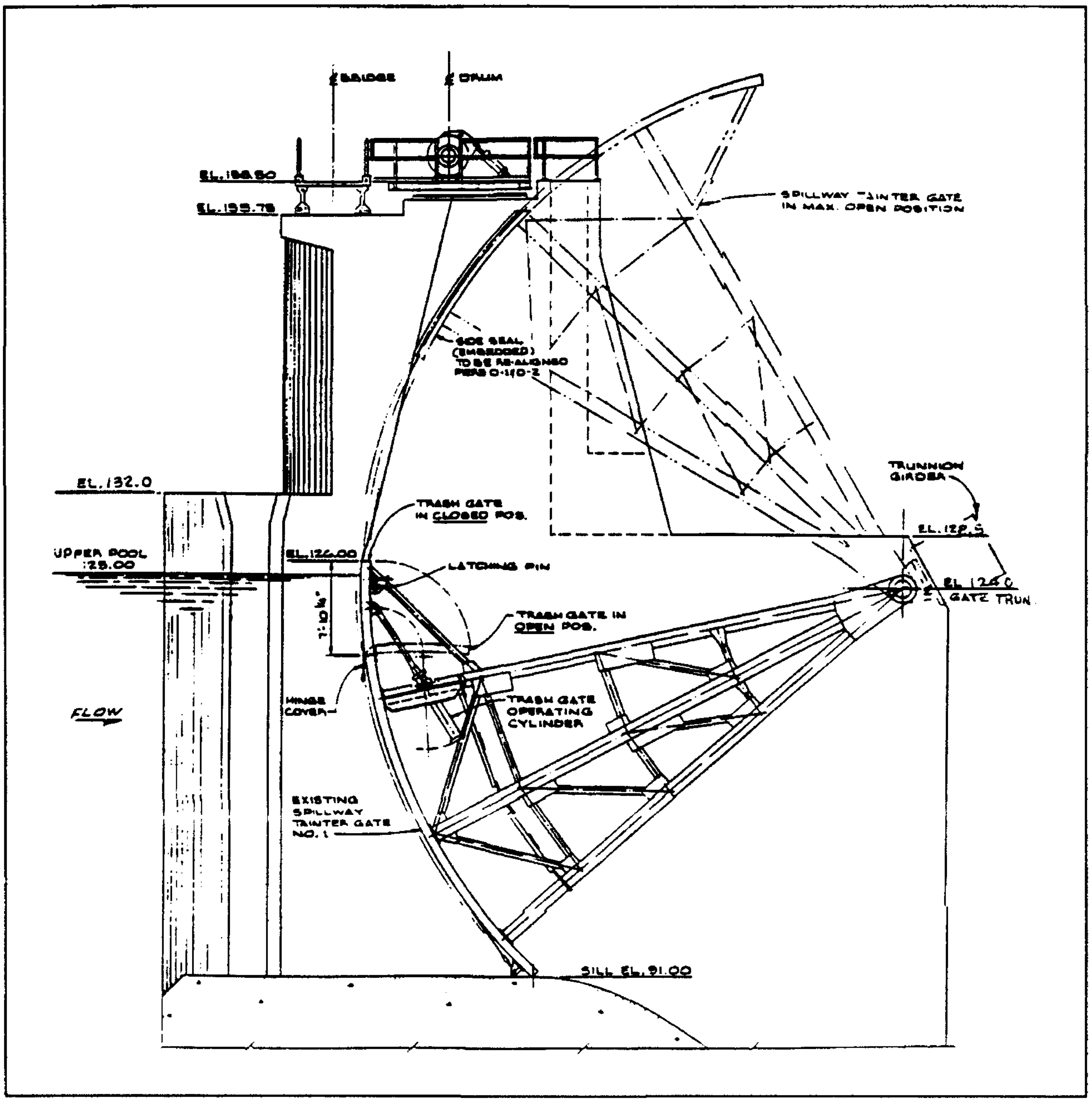

Figure 4. Gated spillway section showing trash gate location

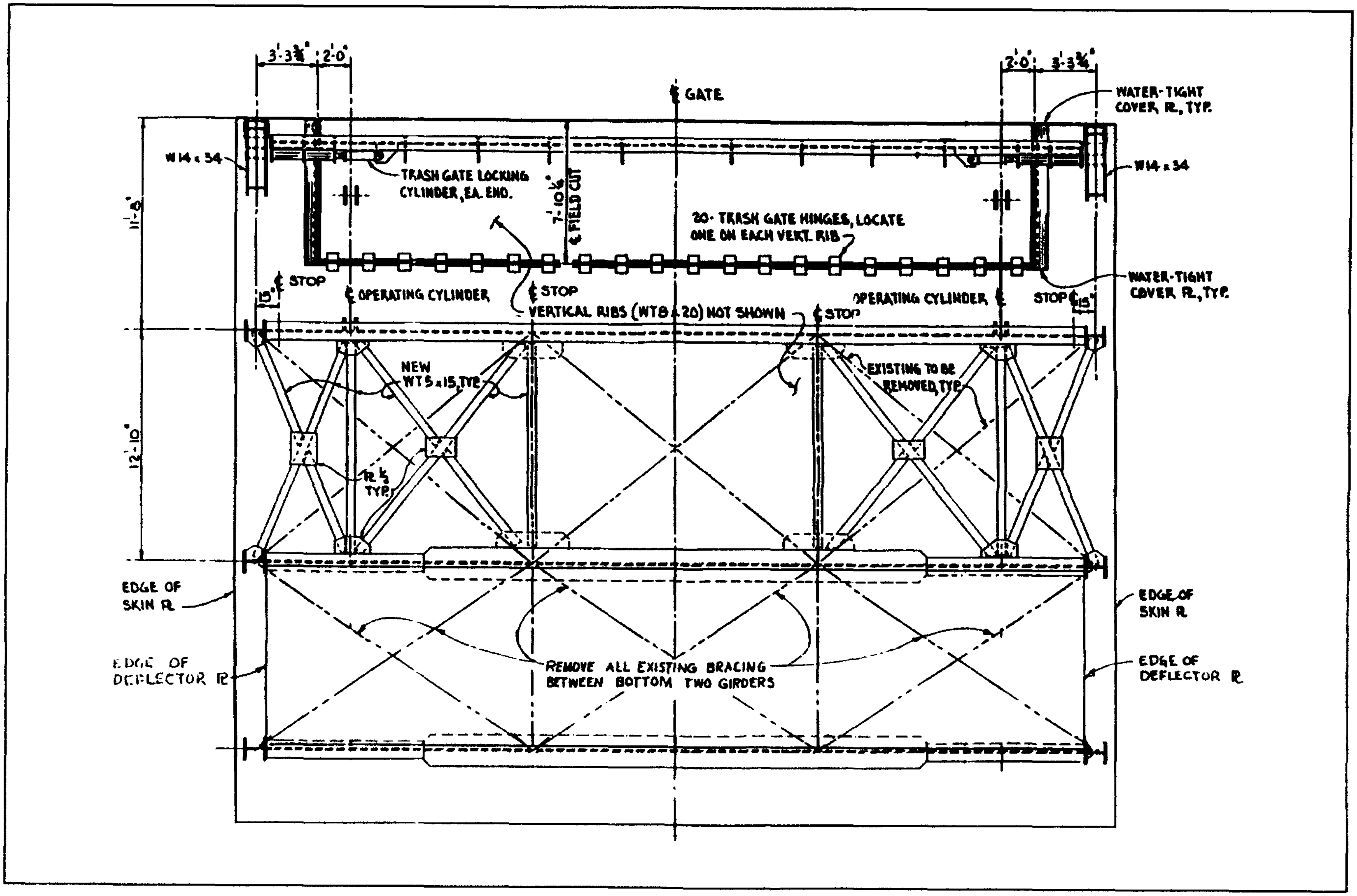

Figure 5. Downstream elevation of tainter gate and trash gate

plate assembly mounted on the downstream face of the ribs on the trash gate (Figure 6).

The trash gate's hydraulic cylinders are operated by a small hydraulic system mounted on the spillway pier. Operation of the trash gate is by a manual four-way control valve. This system is totally independent of the tainter gate system.

Seals are provided on each end and across the bottom of the trash gate for a watertight fit. A fabric reinforced rubber belting material covers the hinged area on the upstream of the trash gate to prevent debris from becoming lodged during the closing operation.

Between the two bottom girders on the downstream flanges, a steel deflector plate was

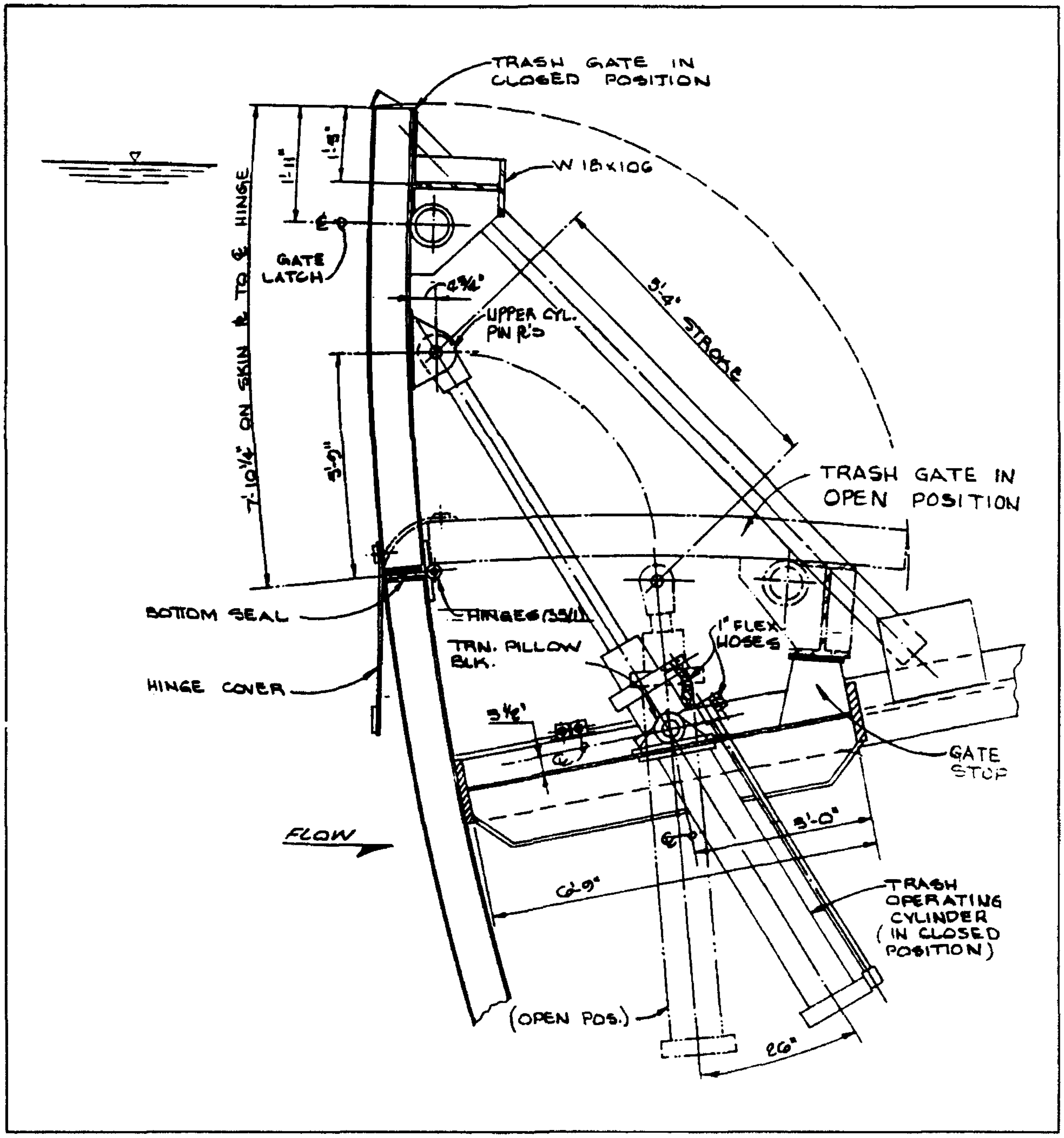

Figure 6. Detailed section of trash gate

added to deflect trash and water downstream during trash gate operation. Deflector plates were also added to the strut arms between the top and middle struts and between the top strut and the diagonal brace supporting the ribs at the lifting cable. The deflector plate between the bottom and middle girders replaced gate cross-bracing. Cross-bracing between the top and middle girders was rearranged to better suit the load from the cylinders and latches. Figures 7 through 10 show the completed trash gate and the initial testing of the trash gate.

A major concern with the project was whether or not the existing tainter gate machinery could lift the tainter gate with the trash gate added. The additional structural members, hydraulic cylinders, seals, and other components added approximately 18,500 lb. Weight control became very important during the design phase. As hoped, this additional weight did not exceed the machinery's safe operating range, and the machinery was able to lift the gate satisfactorily.

Conclusion

The trash gate was put into operation in March 1991. Since then, it has performed extremely well, and there has not been a buildup of trash and debris in front of the turbine intake area. By adding this trash gate, the debris problem should be eliminated at Robert F. Henry Powerhouse. With the success of this trash gate system, other projects with debris problems may want to take a serious look at this innovative means of trash removal.

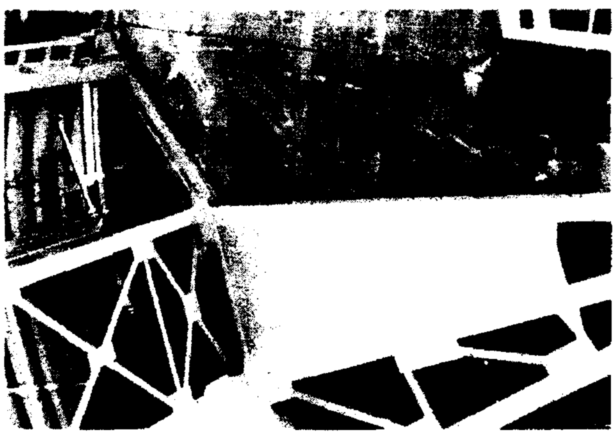

Figure 7. Trash gate in closed position

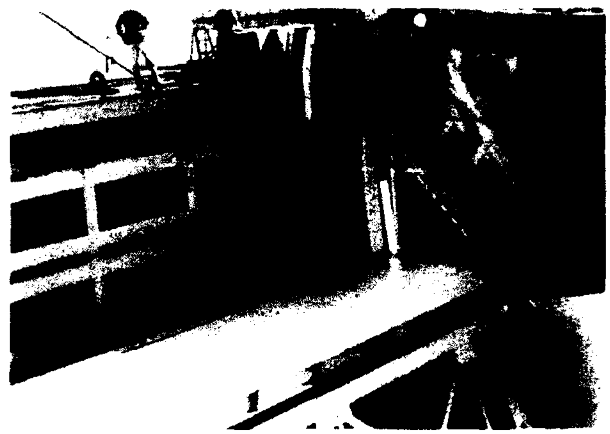

Figure 8. Trash gate in open position

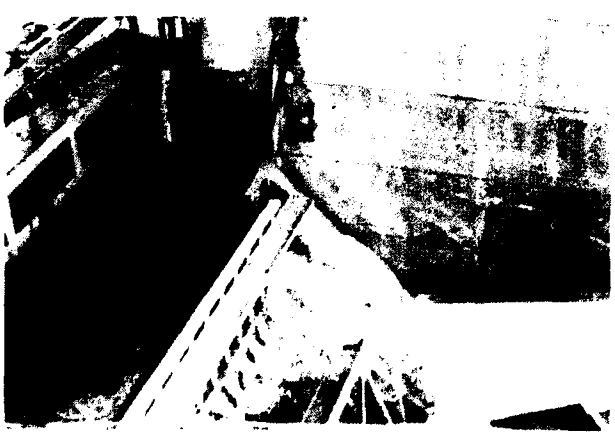

Figure 9. Flow passinq through partially opened trash gate

Figure 10. Flow passing thorugh 75-percent-opened trash gate

Slurry Constructed Diaphragm Guard Wall Bonneville Navigation Lock

by
Jerome Maurseth[1] *and Jeffrey S. Sedey*[2]

Abstract

The US Army Engineer District, Portland, has completed construction of the diaphragm walls contract as phase 3 of the Bonneville Navigation Lock Project. The majority of construction for this phase of the project consisted of slurry constructed diaphragm walls. Discussion within this paper establishes project familiarity and concentrates on design considerations of the slurry construction method, pile fabrication including butt welding, pile installation, tools and tool modifications, other construction issues, and excavation costs.

Diaphragm Walls Contract

The new Bonneville Navigation Lock project is located 40 miles east of Portland, OR, on the Columbia River, just south of the existing navigation lock. Once complete, the new lock will join the first and second powerhouses, spillway, and existing navigation lock at the same location on the largest river in the western US. Construction of the new lock will consist of four phases: the railroad relocation contract (completed), the excavation contract (completed), the diaphragm walls contract (completed), and the main lock contract (underway). Phase 3, the diaphragm walls contract, will provide seepage cutoff, additional slope stabilization, and permanent and temporary walls for the upstream end of the new lock. Various major features include a guard wall and cap, a diaphragm wall, two vertical drainage shafts, and a seepage cutoff wall (Figure 1).

The guard wall is a 3- to 3.5-ft-thick slurry constructed diaphragm wall totaling approximately 104,000 sq ft of excavated area. It consists of steel piles as the main structural members with tremie concrete to act as lagging. Pile sizes range from W36 × 194 on 6-ft centers to W36 × 300 with variable thickness cover plates (up to W36 × 848 equivalent) on 4-ft centers and 130 ft long. Adjacent to the guard wall is a Union Pacific rail line ranging from 55 to 105 ft from the river face of the wall. The guard wall is a permanent structure utilizing a cap beam and permanent soil anchors. The anchors extend as much as 200 ft behind the wall and under the railroad into the hillside. To keep the anchorage for permanent tiebacks above the pool and to provide for future accessibility, only one row of ties will be provided at 2 ft below the top of the wall. The exposed height of the guard wall ranges from 30 ft to 68 ft, which, when supported by only a single row of tieback anchors at the top, requires wall sections of considerable size to meet the design criteria (Figure 2).

The guard wall cap is designed as a continuous beam at the top of the guard wall to transfer loads from the separate panels into

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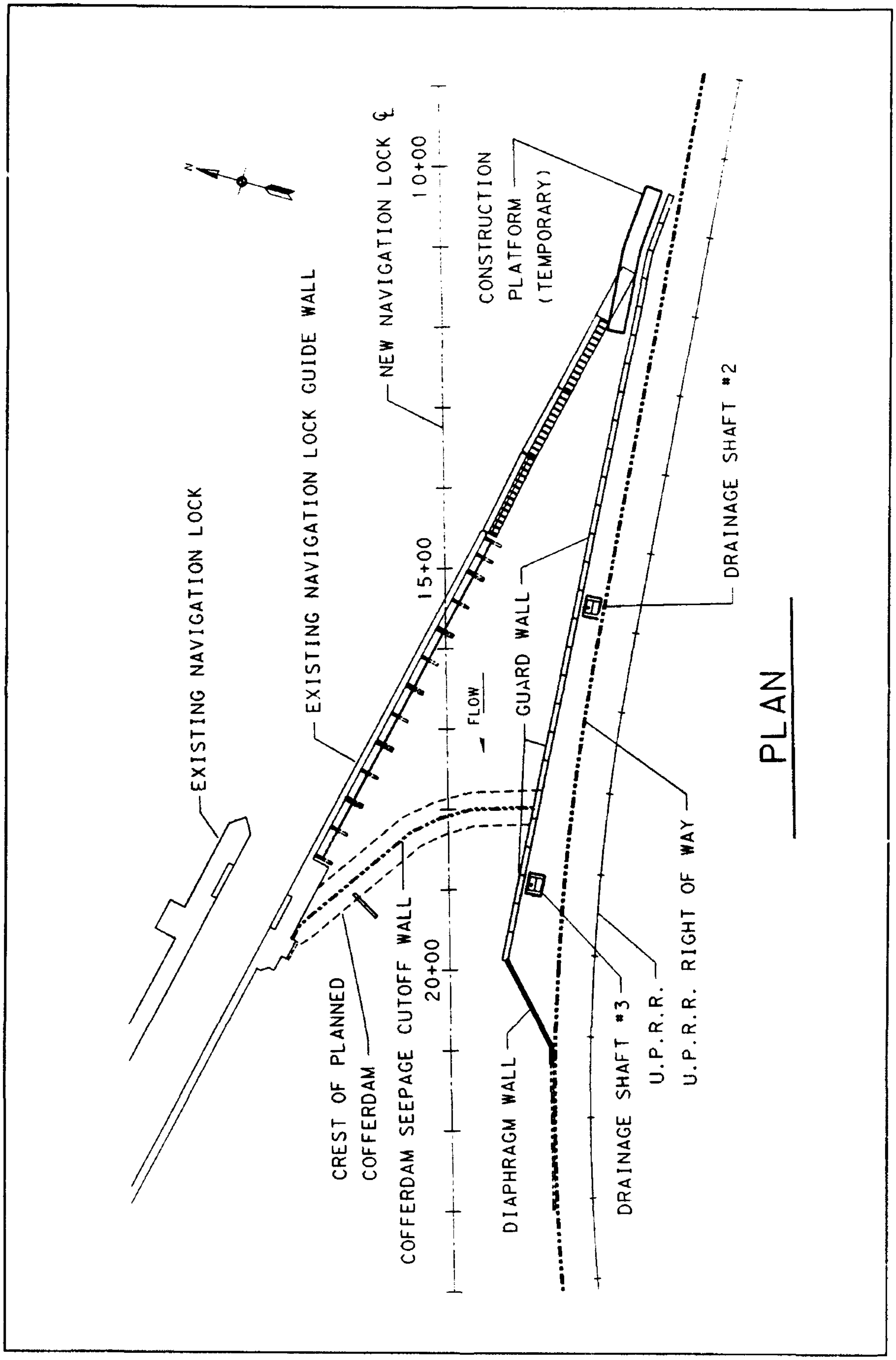

Figure 1. General arrangement plan

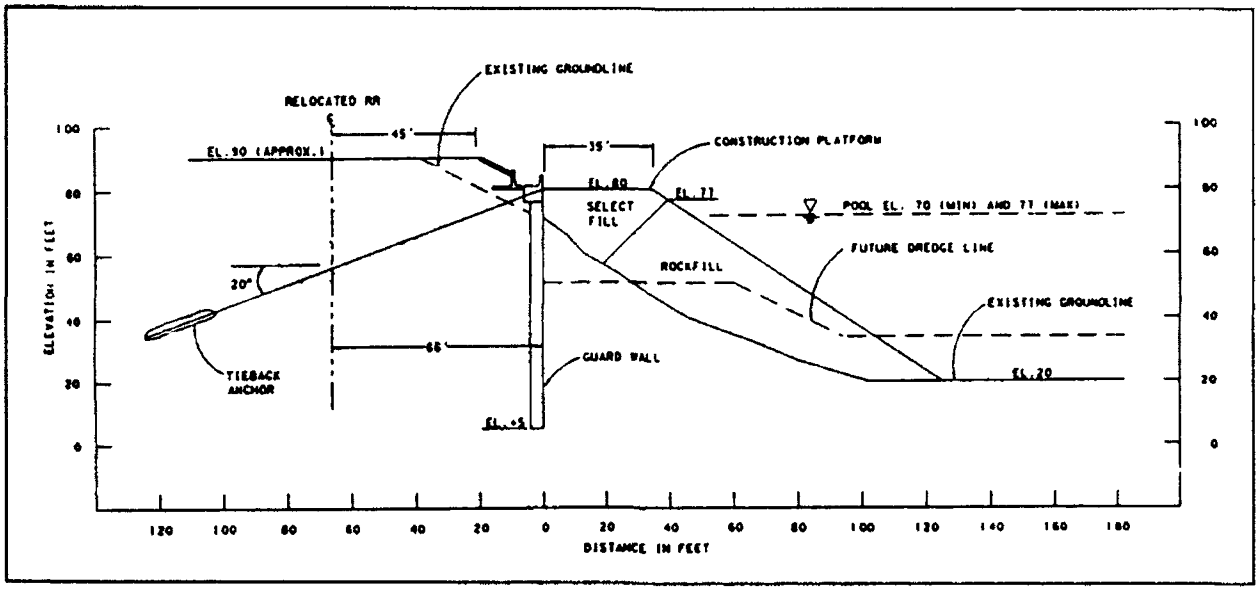

Figure 2. Guard wall section

the tieback anchors. Should a tieback fail, the resulting overstress carried by the adjacent ties will not exceed an allowable overstress. The cap also is required to transfer the loads from panels in front of the drainage shafts to tiebacks on each side of the shafts.

A cofferdam is to be established about 200 ft upstream of the downstream end of the guard wall. With this in mind, downstream of the cofferdam the face of the guard wall will be excavated in the dry to an exposed height of 94 ft. However, upstream of the cofferdam excavation in front of the guard wall can be accomplished by in-water methods only.

The diaphragm wall (Figure 3) is a 3-ft-thick slurry constructed steel pile wall that will have temporary tieback anchors throughout its height. The wall consists of W36 × 260 piles. The cofferdam is located upstream of the diaphragm wall allowing excavation along the face to provide access for installation of tieback anchors and construction of future structures during the main lock contract. The diaphragm wall will serve as a temporary tieback wall, allowing construction of the guard wall monolith and south intake monolith. Once construction of the guard wall monolith and south intake monolith is complete, temporary tieback anchors are neglected, and soil loads are transferred through the diaphragm wall to the monoliths. No permanent use is required of the diaphragm wall; this area will be backfilled once monolith construction is complete. The total square footage of wall face is about 24,000.

Two drainage shafts were constructed in the diaphragm walls contract. The function of the drainage shafts is to provide for additional slope stability to the hill behind the guard wall and to reduce water pressure on the guard wall. To accomplish this, two tiers of horizontal drains will be drilled and installed in each shaft. These horizontal drains radiate 500 ft out from the shafts at each tier. Water carried to the drainage shafts by the horizontal drains will be carried downstream by 18-in.-diam line connected between the shafts. At the downstream end of the guard wall, where the diaphragm wall begins, the 18-in. drain line will pass through the diaphragm wall. The main lock contract will complete taking drain water to the downstream end of the lock.

A seepage cutoff wall was constructed in the planned cofferdam by the slurry construction method. This cutoff wall is 2 ft thick, 60,000 sq ft, and consists of 1,000-psi slurry concrete.

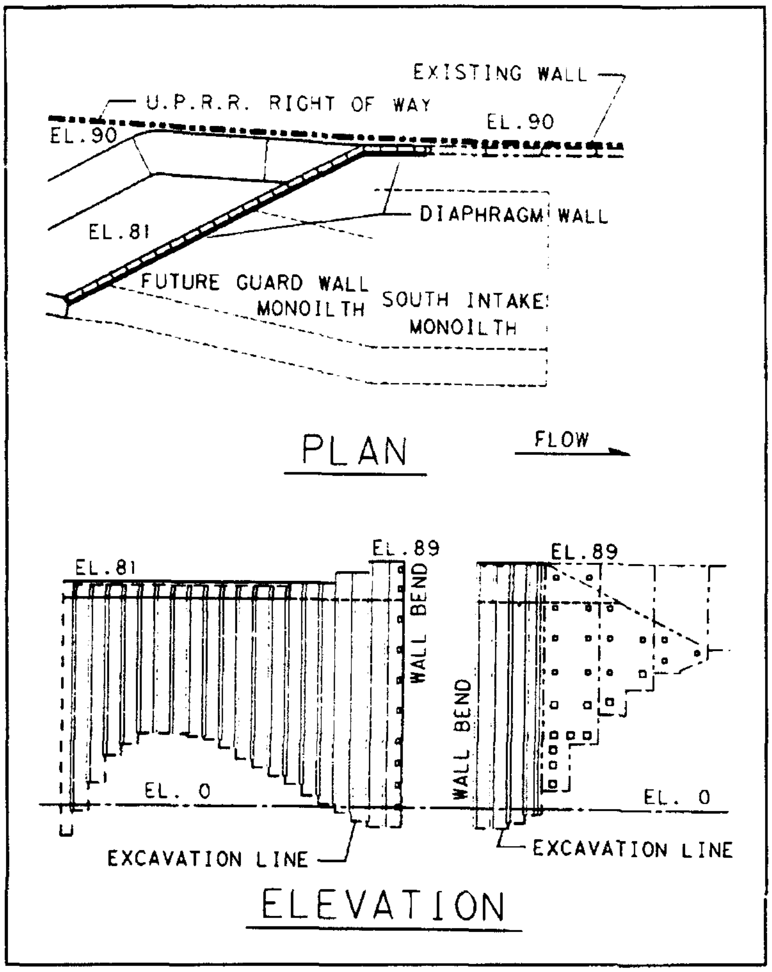

Figure 3. Diaphragm wall plan and elevation

The last major feature of the diaphragm walls contract is the construction platform. Upstream of the cofferdam working surface is limited on the riverside of the guard wall due to the pool. On the other side of the guard wall is the railroad, the closest point being upstream of the cofferdam, minimizing the available work space. At the far upstream end of the guard wall, the wall alignment extends out into what is now pool. Therefore, a platform, known as a construction platform, was constructed to provide sufficient work space for the contractors' equipment. In addition, fill was placed adjacent to the platform to serve as the soil through which much of the upstream end of the guard wall was excavated.

Design Considerations

Slurry construction method

During preliminary engineering of the upstream approach to the new Bonneville Navigation Lock, at least two major problems were apparent. One was the close proximity of the Union Pacific Railroad, and the other was the geologic conditions at the site. Because the railroad is so close to the wall alignment (even after being relocated), laying back the slope and constructing conventional buttress walls or gravity monoliths were not viable options. Further, the abundance of boulders and cobbles precluded the use of sheet-pile cells. In one area, where a slide had been creeping (known as the Railroad Slide), the local soil was removed and replaced by rock fill in the 1940's. One option that did look viable was the slurry construction method used to build structural bearing and retaining walls.

Most slurry wall construction in the United States is for seepage cutoff purposes. This method is to be used again in creating a cofferdam from in situ soils in a limited space at the upstream end of the new Bonneville Navigation Lock. However, it is the use of slurry "diaphragm" walls that made possible the channel alignment at the upstream end of the navigation lock. A slurry diaphragm wall is a structural wall designed for bearing or for retaining soils. The structural charactoristics are what distinguishes diaphragm walls from slurry "cutoff" walls which control seepage and may or may not contain concrete.

The construction process for a diaphragm wall (Figure 4) can have many variations, but typically is as the following description. Guide walls (not shown) are constructed for guiding the alignment of the "clamshell bucket" to be used. The clamshell will excavate a vertical slot known as a "panel." As the excavation proceeds downward, the panel is maintained open by a bentonite water slurry. This slurry, kept at a level a few feet above the local water table, and having a unit weight somewhat greater than water, provides a positive hydrostatic pressure on the side walls of the panel. Stop end tubes are placed at one or both ends of the panel to form guides for the clamshell excavating adjacent panels. Once the panel has been excavated to its designated depth and has been desanded, a reinforcement cage is lowered into the slot and dogged off into place. A tremie pipe is then directed to the bottom of the panel and concrete placement begins.

The concrete must have high flowability and a set time sufficient to allow the place-

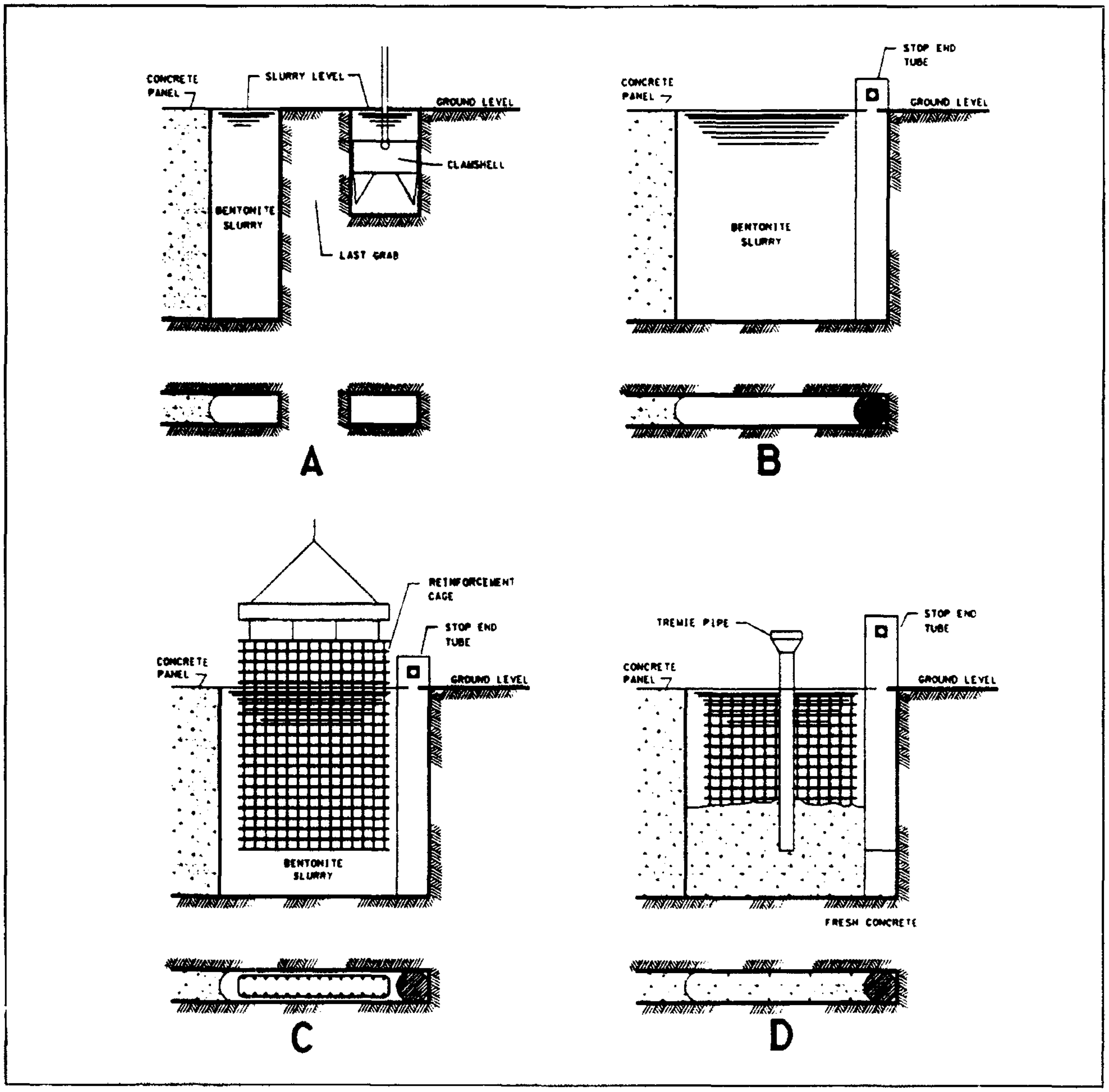

Figure 4. Construction sequence of a diaphragm wall

ment to be completed. It is important that the set time is extended to avoid problems with delayed fresh concrete mixing with already-placed concrete or delivered concrete awaiting placement. As the concrete is tremied into place, the slurry is displaced and is removed at the surface. The tremie pipe is always to remain embedded well below the surface of the concrete to ensure vertical flow of an even surface. It is important that the stop end tubes are not removed too early or too late. Once the panel has been concreted, stop end tube removal may begin. It is not required that the panel concrete placement be completely finished prior to pulling of the stop end tubes; however, the tubes must always be at a depth below where the concrete is still fluid. Therefore, pulling of the stop end tube part way to keep the bottom from becoming stuck with the hardening concrete while the top still retains fluid concrete is common practice. Pulling the stop end tube too soon will allow concrete to slump into the cavity left by the tube. Pull it too late and the tube may become stuck, requiring hydraulic jacks to remove it.

After the first panel is complete, the remainder of the wall is completed by excavating every other panel, termed primaries, and then completing the panels between the primaries, termed secondaries. The round formed ends of the primaries, due to the stop end tubes, are used as guides for the clamshells while excavating secondaries. An alternate and commonly used procedure is to construct the first primary and then construct adjacent panels sequencially using the previous panel as the guide shown in Figure 4.

An alternate to a reinforced concrete wall is to use steel piles lowered into the excavated slurry trench. Lowered singly or in pairs laced together to maintain verticality of the individual piles, the concrete tremied in place between the piles serves as lagging. The flanges of the piles also serve as guides for the clamshell excavating secondaries, thus eliminating the need for stop end tubes. This type of wall was used at Bonneville for both the guard wall and the diaphragm wall (Figure 5).

For design of slurry diaphragm wall members, certain recommendations have been followed (Xanthakos 1979). In general, the recommendations are for taking a conservative stand when dealing with the uncertainties of slurry concrete walls. First of all, a 15-percent reduction was taken for the design strength of the concrete. Secondly, generous lap lengths of 1.5 to 2.0 times the normal splice lengths are recommended for concrete reinforcement. The laps for this project were increased by a factor of 1.8, which was applied to the development length.

Other recommendations concern the constructability of slurry wall panels. Because concrete must flow through and around reinforcement and displace the bentonite slurry, it

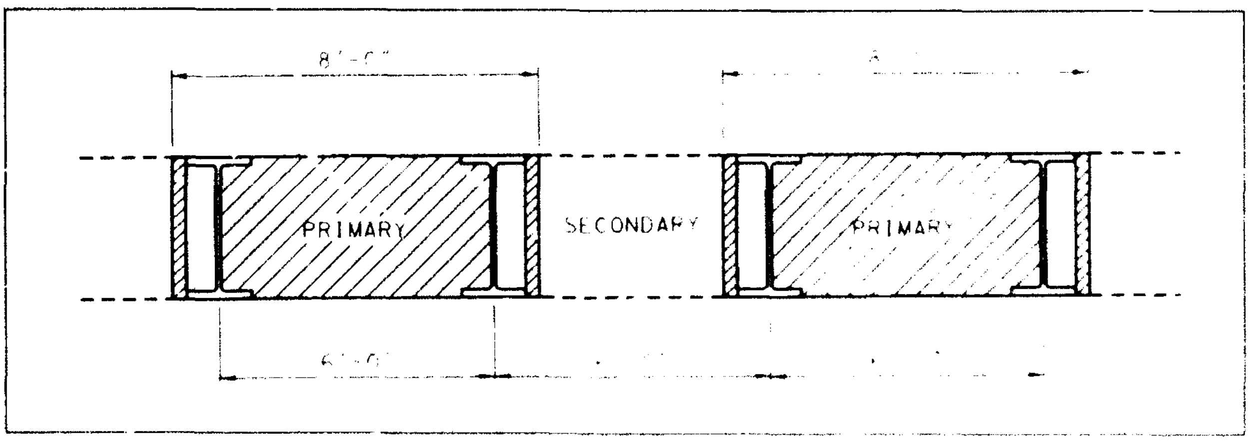

Figure 5. Diaphragm walls with piles

is important that the flow not be restricted or the restrictions be minimized. Therefore, it is important that spacing of reinforcement be maximized. For vertical reinforcement, the minimum spacing recommended is 6 in., preferably 9 in. For horizontal bars, the minimum spacing recommended is 12 in. In addition to good bar spacing is the flowability of the concrete. One measure of flowability is its slump. A slump of 7 to 8 in. minimum is recommended. We had measured slumps of 9 to 10 in. during concrete placement in the diaphragm walls contract. Spacing of the reinforcement also must allow for insertion of a 6- to 10-in. tremie pipe between bars. Some contractors have said joints on the tremie pipes may be oversized and may require up to 14 in. of space, but this was not observed.

Geology

The geologic history of the Bonneville project area is long, complex, and very interesting. However, there is insufficient space in this paper to provide a detailed background.

We dealt with several geological problems unique to this project location. One problem concerned past known slides and the potential for reactivating them. Another concern was the prevalence of boulders throughout the overburden material.

In the process of excavating panels for the guard wall, the panels would be aligned perpendicular to the direction that the potential slides would tend to travel. The opening of a long trench would be equivalent to removing the toe of a known slide. Therefore, no backhoe trenching or a series of open slurry wall panels was allowed. Especially true in the railroad slide area was the requirement for allowing no more than one panel to be open in four or five panel lengths. Panel widths were limited to 20 ft perpendicular to the slide direction ensuring that arching occured around open panels.

The use of clamshells to excavate the slurry wall panels required a certain amount of rock chiseling to remove boulders larger than the thickness of the walls. Therefore, in soils that are known to have boulders, it may be an economic advantage to use a wider bucket. A 3-ft bucket width rather than a 2-ft width (thus wall thickness) will allow for a greater number of small boulders to be removed by the clamshell before chiseling is necessary. This is always open to evaluation by the estimator, since a larger bucket may require a larger crane and other added costs.

Boulders also create holes or pockets in the side walls of the panels, requiring extra concrete placement. In the excavation contract, 11 panels were constructed with about a 30-percent concrete overrun.

Construction Considerations

Pile fabrication and panel tolerances

By nature of the excavation method, the width of a slurry trench is equal to the width of the clamshell bucket used to excavate it. If a 36-in. panel is selected in design, the contractor will construct guide walls spaced at approximately 38 in. to accommodate a 36-in. clamshell. During the excavation, the clamshell will incidentally scrape and gouge the panel sidewalls such that the final panel width is at least 1/2 in. greater than the 36-in. design widths. How much that over-width actually is greatly depends on the soil type being excavated. Ideally, it would appear that a 36-in. steel pile would have little problem fitting into 36-in. design width panel. However, this is where pile fabrication, installation technique, and tolerances become important factors.

According to the American Society for Testing and Materials (ASTM) A6, rolled steel shapes are allowed permissible variations in their dimensions. Among those variations listed are camber and sweep. For "W" shapes, the permissible variation (inches) for camber and sweep is: 1/8 in. × (number of feet of total length/10). Therefore, for a 130-ft W-section, the permissible camber and sweep is 1-5/8 in. Since shapes are rarely rolled this

long, the longer piles required splicing, to which the permissible camber and sweep was applied to the total length. For our W33 × 241 with 1-in. cover plates, the total depth plus camber could be as much as 37.8 in., exceeding even an "overwidth" 36-in. panel.

Several solutions remain open for resolving this conflict. First and most obvious would be to use the next size larger clamshell bucket. This will work, but there are tradeoffs. As the clamshell width increases (6-in. increments) so does the weight of the clamshell, the excavated material (weight per bite and total quantity), and possibly the working crane size. An alternative is to weld mining bits to the side surfaces of the clamshells which will add 1-1/2 in. to the clamshell width. With the additional scraping and gouging during normal excavation operations, the panel will be even wider.

Similarly, special "scrapers" were developed to achieve the required panel width. For this project, these scrapers consist of 36-in.-diam pipes with plates welded to the pipe sides at the bottom to form a 14-ft by 3-ft box shape. Welded to the outer surface of the plates were the mining bits. Continual working of the clamshell bucket, sometimes alternating with scrapers, and finally scraping the completed panel usually achieved a panel wide enough for a smooth fit of the piles. Panels were checked by lowering a single pile into each pile location, ensuring no hangup would occurred.

Installation

Once panel excavation is complete, piles are lifted and placed into the slurry trench one at a time. After the piles are placed, the crane would lift either two or three piles at once with a lifting beam. The piles would then be connected by 3/4-in. by 6-in. plates welded on the inside flange surfaces every 10 to 15 ft of pile length. Several purposes were served by installation of these tie bars or "lacing." First, the appropriate spacing of the piles relative to each other was maintained. This also eliminates concerns of sweep in the piles. Second, correct relative spacing of the piles ensured proper placement of reinforcement cages in between. Third, the piles would now act as a unit during concrete placement. The unit would not be as easily moved around by concrete placement. Also, the piles in the slurry walls, if not laced, would easily be misaligned, turned, or bowed during concrete placement. The pile surfaces, later to be exposed for installation of multiple rows of tiebacks, will provide more uniform flange surfaces for attaching the walers. This was observed as especially true when compared to an adjacent soldier pile wall installed by drilled-in piles.

Pile splicing

Several sections of the guard wall required pile sizes equivalent to "jumbo" categories group 4 and group 5 for tensile property classification. These would be piles greater in size than W36 × 300 or W33 × 291, for instance. Additionally, the length of each pile was approximately 130 ft. Because the section sizes required (up to W36 × 848) are not available domestically, and only domestic steel is to be used, equivalent piles were fabricated by butt splicing two W36 × 300's and attaching cover plates. Technically, a W36 × 300 is not in a group 4 or group 5 classification. However, since it was marginal, the butt splicing was treated as though it was in the jumbo categories.

Special consideration for welded butt splices (bolted splices were not a viable option) are necessary. Discussion of these considerations is provided in much more detail in an accompanying paper to be presented at this conference, "Finite Element Modeling of Welded Thick Plates for Bonneville Navigation Lock." Therefore, they will be addressed only briefly here.

Primary concerns address initiation of cracks either in the web or flange welds during cooling or at the access holes where the web and flange meet. Many reasons can be given for how the cracks are initiated. The primary preventative actions are to establish the joint heating procedures, to specify the toughness characteristics for the base material

and the weld, and to provide appropriate access holes at the flange-web intersection.

The American Institute of Steel Construction (AISC)(1989) now addresses these concerns, beginning with Specification Section J1.7 and makes specific recommendations. Prior to issuance of the ninth edition of the AISC manual, the toughness requirement specified for our pile splices was 15 ft/lb at 40 °F by the Charpy V-notch test. AISC now recommends 20 ft/lb at 70 °F. Interestingly, the toughness requirement specified could not be met by an electroslag/electrogas welding process but was easily passed by the submerged arc welding process. Review of the electoslag process was being conducted to determine how it might meet the toughness requirements since it has significant economic advantages.

Other construction considerations

Additional concerns were raised throughout construction of the slurry walls:

- In soils with significant subsurface variations, exploratory drilling on 25-ft centers along the wall alignment for soil sampling has been recommended. This contract had exploratory drilling on 50-ft centers. Still, a changed conditions claim on the bedrock profile was filed, and although the claim has not been settled, the final amount may be substantial. The importance of accurate subsurface soil interpretation which is supported by an appropriate drilling and investigation program must be emphasized.

- Because bedrock would likely vary from its expected profile, steel piles required to be socketed into bedrock were ordered with an additional 2 ft. If bedrock varied by more than that, splicing would have been required.

- To expedite excavation through a very dense, tough layer of granular soil, resistant to the clamshell bucket or chiseling, a drilling template was fabricated (approximately 80 ft long) to accommodate minor blasting. After blasting, clamshell excavation with some chiseling was able to continue at a greater excavation rate. Blasting was not allowed at or near bedrock where the walls were socketed into.

- Quality control (QC) on pile alignment to meet the required tolerances was verified by using a slope-inclinometer on every pile installed, before and after concrete placement. Grout tubes attached to every pile installed were used for the slope-inclinometer. Piles installed on the first panel excavated utilized minimal lacing. After concrete placement, a check of the pile alignments showed unacceptable results. Lacing was used for a significant depth on the piles from then on with results complying with the specified tolerances.

- When the concrete is tremied into place, it is important that adjacent panels do not have different concrete levels, creating a differential head. A differential head can lead to several problems, including pile misalignment and concrete mixing with the slurry. Multiple panel concrete placement (as many as five panels at once) can be accomplished with very careful attendance to differential head and a ready supply of fresh concrete.

Excavation Cost Estimates

Table 1 consists of selected line items from the abstract of offers-construction for the Diaphragm Walls Contract. These are "installed" estimates, without attempting to detail the specific requirements of measurement and payment for each line item. Table 1 is worthy of getting a "feel" for the costs involved with providing slurry walls installed in difficult soils.

Conclusions

The slurry construction process offers a viable economic solution to many problems. There may be special considerations required

by the nature of the construction process. However, many unique situations can be resolved by equally unique constructable solutions and with diligent quality control produce a high-quality product.

Table 1
Diaphragm Wall Contract Estimates

Description of Offered Items	Est. of Quantity	Unit	Gov't Est	Contractor Unit Price No. 1	No. 2	No. 3
42-in.-wide slurry trench, in overburden	17,500	sq ft	$65.10	$80.00	$80.00	$40.00
42-in.-wide slurry trench, in bedrock						
First 3,500	3,500	sq ft	139.60	150.00	150.00	300.00
Over 3,500	3,500	sq ft	139.60	150.00	150.00	300.00
36-in.-wide slurry trench, in overburden	85,800	sq ft	100.47	85.00	60.00	30.00
36-in.-wide slurry trench, in bedrock						
First 3,200	3,200	sq ft	208.00	130.00	130.00	270.00
Over 3,200	3,200	sq ft	179.40	45.00	45.00	270.00

References

American Institute of Steel Construction. 1989. *Manual of Steel Construction*, 9th ed., New York.

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McAlpine Lock Replacement Project
McAlpine Locks and Dam

by
Veronica L. Rife, PE[1]

Abstract

The Ohio River Mainstem Study McAlpine Interim Navigation Feasibility Report by the Louisville District recommends the construction of an additional 1200-foot lock at the Louisville, Kentucky, site. Coupled with the existing lock, this would result in two 1200-foot locks in service. The project features include the new lock with a bascule bridge over the lock, a fixed weir with roadway on top to connect the bridges over the two locks and upper approach canal wid ning. Engineering and design studies to establish the details of the project plan are currently underway. The project Planning, Engineering and Design (PED) Activities feature two main project Feature Design Memorandum, Lock and Canal Approach, leading to Plans and Specifications for the project. Additional Feature Design Memorandum will include Construction Material and Structural Properties DM's. The District is currently requesting a General Design Memorandum waiver for the McAlpine project. Total PED activities will span 5 years.

Introduction

The existing McAlpine Locks and Dam is located on the Ohio River, 604.5 miles downstream from Pittsburgh, Pennsylvania, at the Falls of the Ohio rapids, the only major natural obstacle to navigation on the Ohio River. One-third of all current waterborne traffic on the Ohio River passes McAlpine Locks and Dam. The McAlpine project has become a conglomeration of different components of locks, and dam and canal systems built at different times from the 1830's on. The most recent work included the construction of a 1200-foot lock and rehabilitation of the auxiliary 600-foot lock in the early 1960's. The existing locks structures consist of a main 110-foot by 1200-foot lock, an auxiliary 110-foot by 600-foot lock, and an inoperable 56-foot by 360-foot lock. The locks are located on the south bank of the Ohio River at the downstream end of a 2-mile canal. The dam structure is formed by components, beginning at the southern end of Shippingport Island and continuing with the Louisville Gas and Electric Company hydroelectric power plant, four lower tainter gates, a 6400-foot section of concrete fixed weir running parallel to the river to the five upper tainter gates and the upper 1000-foot section of fixed weir that run perpendicular to the river and extend to the Indiana shore. These structures extend between Ohio River miles 604.0 to 608.0 in the heart of the greater Louisville, Kentucky, metropolitan area. The McAlpine project is within an area recently established as the Falls of the Ohio National Wildlife Conservation Area. Figure 1 shows an aerial photo of the project site.

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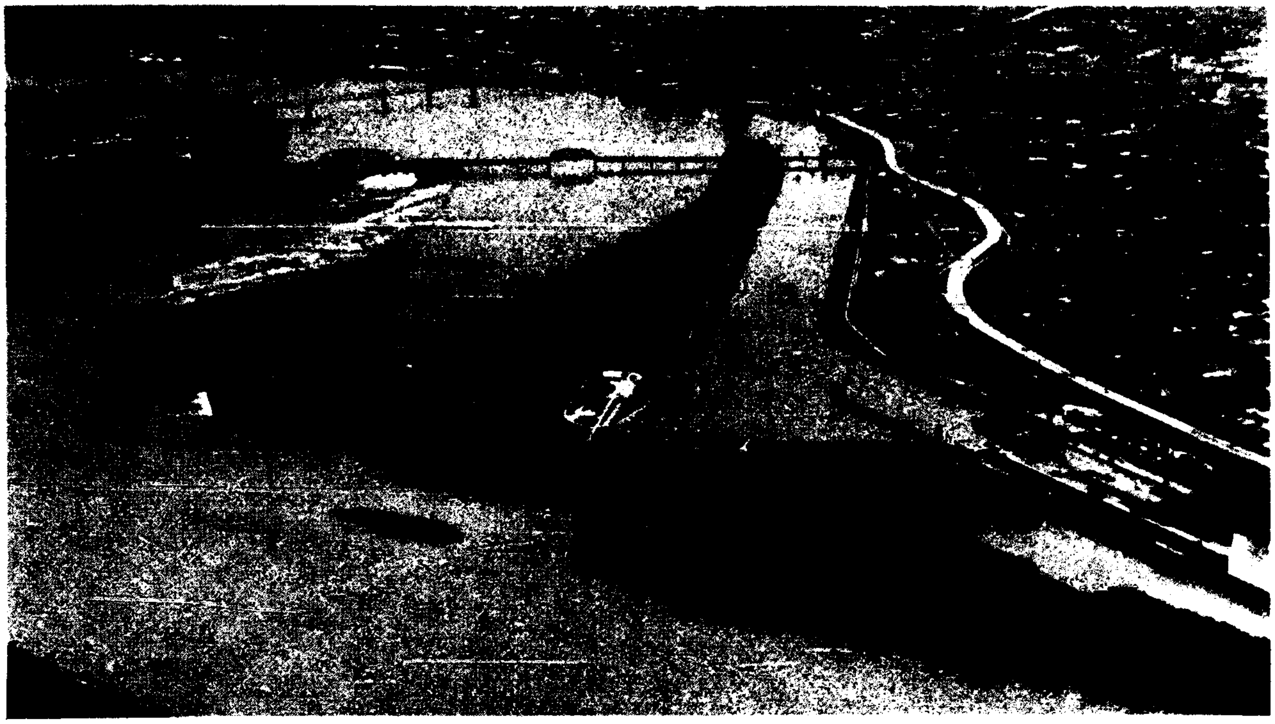

Figure 1. McAlpine Locks and Dam at Louisville, Kentucky

In 1986 commercial traffic demand at McAlpine Locks and Dam was 53.4 million tons and is projected to increase to 72.0 million tons by the year 2000 and 141.6 million by the year 2050. Because of the key strategic location of McAlpine on the inland system, as shown in Figure 2, a scheduled or unscheduled shutdown of the main 1200-foot lock and subsequent sustained operation of the auxiliary 600-foot lock will lead to substantial delays. In June and October of 1987, for example, McAlpine experienced 35 days of such closure in which 723 tows were delayed an average of 30 hours per tow, at a cost to industry of $4.4 million dollars.

Under basic authority from a resolution of the US Senate Committee on Public Works of the US Senate, dated 16 May 1955, and a resolution of the Committee on Public Works and Transportation of the House of Representatives, dated 11 March 1982, the Louisville District studied McAlpine under the Ohio River Main Stem Study to improve or replace "obsolete and inadequate facilities that impede the orderly flow of increasing commerce" on the Ohio River. Several structural and non-structural alternates were evaluated in the study. The plan selected and recommended in the McAlpine Interim Navigation Feasibility Report is Plan B, the construction of a modern 110-foot by 1200-foot lock in the area of the existing 110-foot by 600-foot lock. The new lock would provide reduced transportation costs to the nation, safe and dependable commercial navigation through the year 2050, and minimize impacts to and preserve the environmental resources of the Falls of the Ohio WCA. In November 1990, President George Bush signed into law the Water Resources Development Act of 1990 which authorized the Army Corps of Engineers to build a new 1200-foot lock in the vicinity of the existing McAlpine locks.

Project Description

The McAlpine Lock Replacement project will feature a modern 110-foot by 1200-foot lock to be built in the vicinity of the existing 110-foot by 600-foot lock and 56-foot by 360-foot lock. The new lock would be parallel to the existing 1200-foot lock, with the upper gates located at approximately the same location of the existing lock upper gates. The

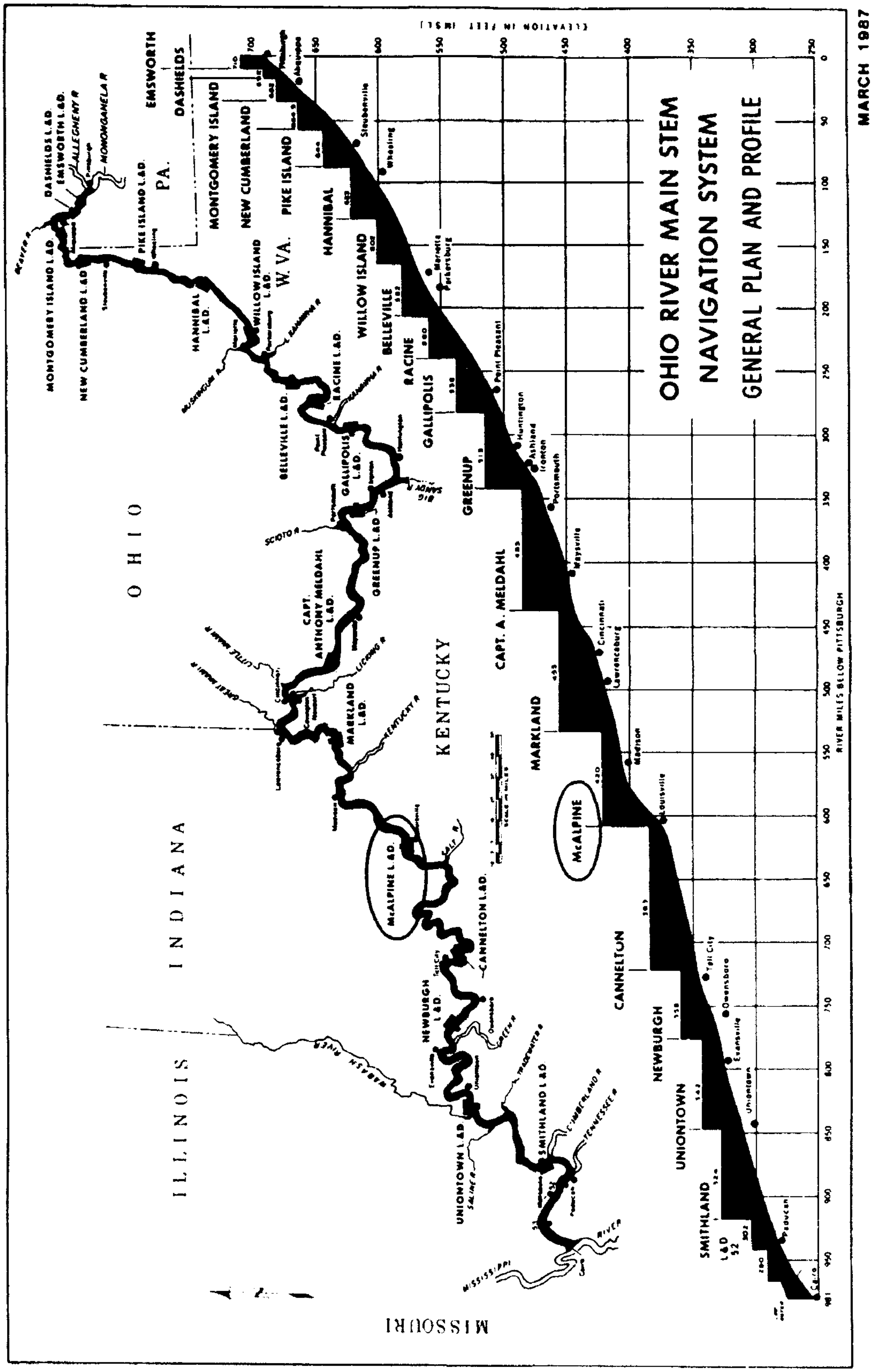

Figure 2. Ohio River main stem navigation system

existing 1200-foot lock was used as the prototype for the new 1200-foot lock structures and filling and emptying system.

The new lock monoliths will be concrete gravity walls founded on strong Jeffersonville limestone. This formation is a strong, resistant type of rock that occurs as the bedrock strata in the Falls of the Ohio. The top of the lock walls will be 79.0 feet above the floor.

Horizontally framed miter gates with single skin plates will be provided at each end of the lock. An emergency vertical lift gate will be provided upstream from the upper miter gates. The culvert valves will be reverse tainter gates. Approach walls, ranging from 160 feet to 1200 feet in length, will be designed as a concrete gravity wall structure supported on cellular sheet pile cells and be provided upstream and downstream of the lock. The lock filling and emptying system will employ a split lateral system. An existing discharge structure and stub outfall culvert intended to accept flow from a new lock landward of the existing 1200-foot lock will be used.

A new bascule bridge spanning the upstream end of the new lock and a fixed weir with a two-lane roadway on top would replace the existing swing bridge that currently spans the 360-foot and 600-foot locks. To facilitate navigation of modern tows into the new lock, the canal approach just upstream of the lock entrance would be widened. The remaining geometry of the canal will be sufficient for the traffic flow in the study period. The current normal pool of 420.0 would be maintained above McAlpine Lock and Dam. Figures 3 and 4 show the plan and elevations of the lock replacement project.

Before construction begins on the new lock, the existing 1200-foot lock will undergo major maintenance to help minimize possible downtime as there will not be a back-up lock available during such construction. Another alternate plan studied in the Feasibility Study provided partial availability of the auxiliary 600-foot lock during construction. Input from towing industry officials determined that the long-term advantages of chamber separation of the selected plan outweighed the short-term availability of the auxiliary lock during construction in the alternate plan. Construction cofferdam will utilize circular sheet pile cofferdam cells and arcs and the land wall of the 1200-foot lock which will be tied down with rock anchors to provide stability. Public access to Shippingport Inland during construction will be limited. Alternate means of access such as by ferry or by a temporary road across the cofferdam will be investigated.

Rock and concrete removal will be by controlled blasting designed not to damage nearby residential structures as well as the adjacent lock. Construction activities will occur within the area of the existing auxiliary locks and the approach area with spoil disposal to occur between the two 1200-foot locks and on Shippingport Island in an area currently of low habitat value.

PED Activities

The McAlpine project was part of the Initiative '88 and is under the new Life Cycle Project Management Program. The project Planning, Engineering and Design (PED) activities began after receiving Division Engineers notice in January 1990. PED activities include hydraulic studies and navigation model studies, geotechnical studies, thermal studies, and seismic studies. The hydraulic studies will evaluate items such as filling and emptying systems and sill depth for the new lock. Navigation model study will be conducted at Waterways Experiment Station. The 1:70 model currently being constructed will study the lock approaches and obstacles, effects of surge in the canal basin, and the lock emptying system. The model will extend from the Clark Memorial Bridge in front of the Louisville Waterfront to the K&I Railroad Bridge downstream of the locks. Thermal studies, to be conducted by WES, will be presented in a Structural Properties Feature Design Memorandum. Seismic studies will include geological-seismological assessment of the project area to determine earthquake

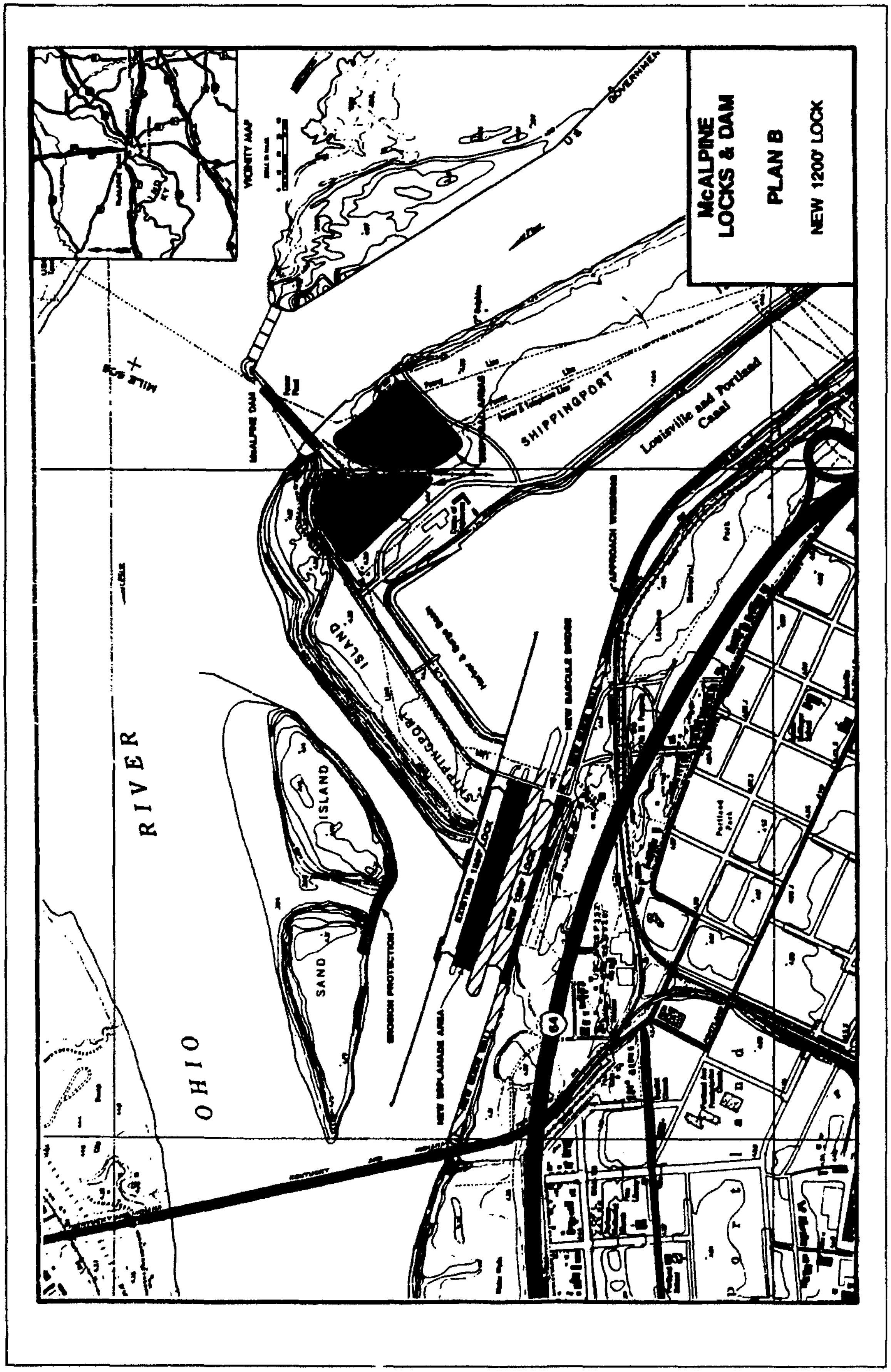

Figure 3. McAlpine Locks and Dam, Plan B

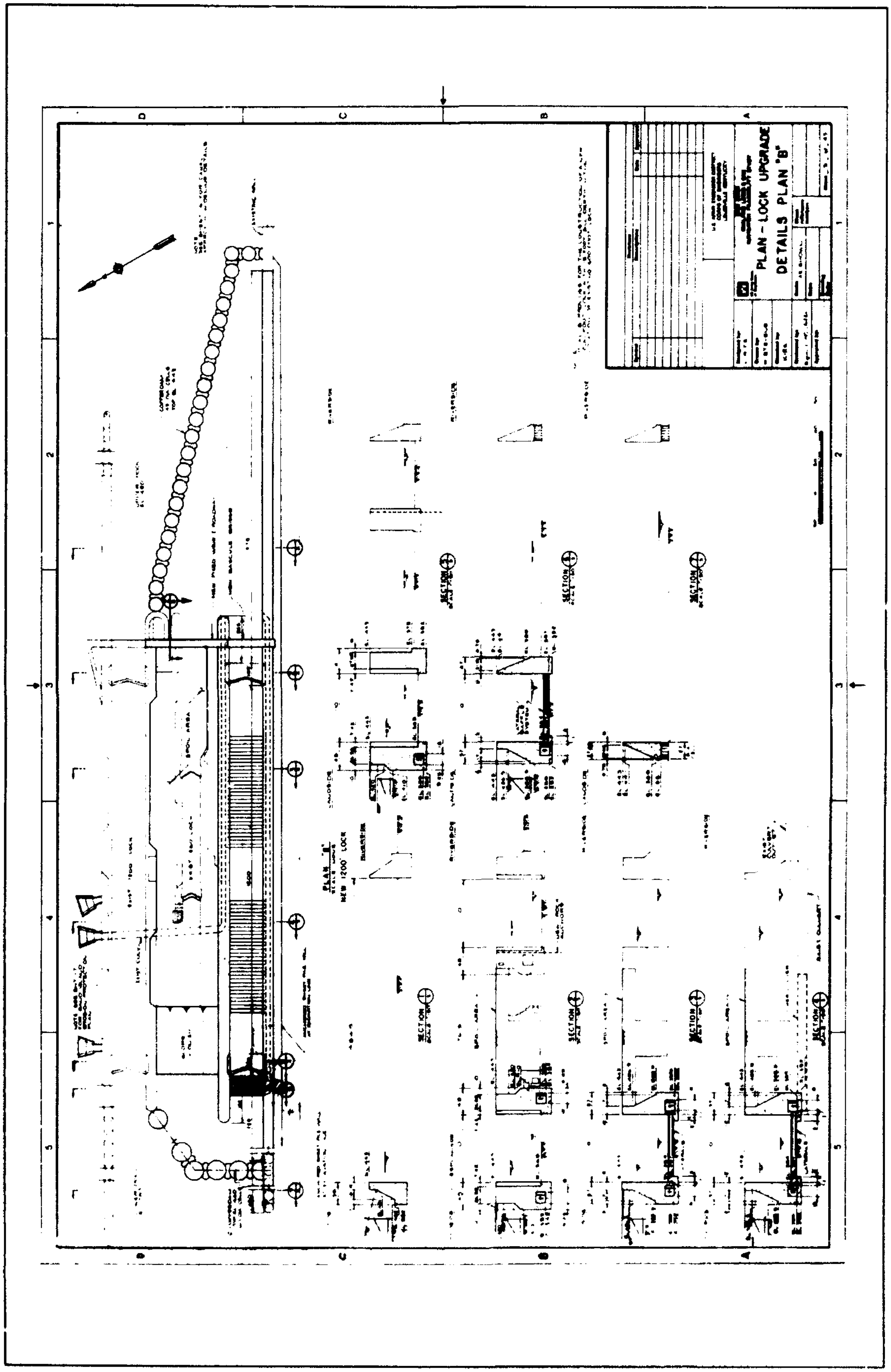

Figure 4. Lock upgrade details, Plan B

motions that can be expected at the McAlpine site, a seismic zone 2. If the studies determine that the site specific peak ground acceleration is a 0.15g or greater, then dynamic and pseudo-static analyses will be performed. Otherwise, only a pseudo-static analysis will be necessary.

The PED studies presently underway in the Louisville District will lead to the preparation of a General Design Memorandum (GDM). The GDM is scheduled for completion in September 1992. If the District request of a GDM waiver is approved, the PED studies will be presented in a Lock Feature Design Maintenance scheduled for completion in FY 1994.

A midpoint Technical Review Conference to be scheduled in Fy 1992 will provide information and cost estimate to date and provide data for Value Engineering studies on the project.

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US Army Engineer District, Louisville. 1990 (Jan). Ohio River Mainstream Study McAlpine Interim Navigation Feasibility Report, Volumes 1-4, Louisville, KY.

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Construction of the San Antonio Flood Tunnels

by

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Abstract

The tunnels are located in San Antonio, TX. They consist primarily of deep tunnels to divert flood waters beneath much of the more highly developed areas of the city, thereby providing flood protection for these areas. This allowed more options open to the city regarding the development of the surface channels and adjacent areas. The tunnel projects involve two separate tunnels with two separate sets of intake and outlet structures. The San Antonio River Tunnel is approximately 3.1 miles in length and follows the general course of the San Antonio River above. The San Pedro Creek Tunnel is approximately 1.2 miles in length and follows the general course of the San Pedro Creek above. Both tunnels have 24 ft 4 in. inside diameters composed of 12-in.-thick precast concrete segments. The tunnels were excavated by a shielded Tunnel Boring Machine (TBM). This paper provides a physical description of some of the construction problems encountered while driving these tunnels with the TBM and installing the precast concrete liner. The solutions to these problems will also be presented.

Introduction

The Congressional Authority for the construction of the San Antonio Channel Improvement Project is contained in the Flood Control Act of 1954, approved 3 September 1954 (Public Law 780, 83rd Congress, 2nd Session). The high cost and disruptive effects of channel modifications in the downtown area of San Antonio led to investigations seeking a more acceptable alternative. The San Antonio Tunnels are the results of the investigation that would satisfy such objectives as (1) flood protection, (2) maximize the benefit-cost ratio, (3) minimize impacts on the existing river walk area, (4) minimize the magnitude and time of disruption of normal activities in the downtown area, and (5) maximize compatibility with local plans for future development.

The overall flood protection project will be constructed in four phases. The actual construction of the tunnels and shafts are in Phase II, titled "Tunnels and Shafts". This phase was scheduled to be completed in March 1990. However, due to the considerable tunneling problems encountered (primarily on the SART), the projected completion date for Phase II is January 1993.

The two flood tunnels of San Antonio, TX, are the San Pedro Creek Tunnel and the San Antonio River Tunnel. The San Pedro Creek Tunnel was completed in July 1989. The San Antonio River Tunnel is currently under construction and has been saddled with numerous delays because of construction problems. Both tunnels have a finished diameter of 24 ft 4 in. The original design for the tunnels

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called for a 24 ft 0 in. finished diameter; however, the contractor was allowed to increase the diameter because of the availability of a hardrock TBM that could be modified to drill the tunnels in the clay shale materials. A smaller diameter tunnel could have been used on the San Pedro Creek Tunnel, but it was more economical to use only one TBM (all things being equal) that could be used to drill both tunnels. Both tunnels are drilled approximately 150 ft below the ground surface. The two tunnels were started at the outlet shafts and driven upstream to the inlet shafts.

General Description

Inlet and outlet shafts

The excavations for the shafts were continually braced through the overburden and the weathered primary materials. Tangent piles were drilled in a circular pattern around the outlet shafts and served as lagging for inner steel ring beams that were spaced at 5 ft on center vertically. The excavation through the unweathered primary was braced with shotcrete and wiremesh. The transition portion of the shafts were also braced with shotcrete and steel ring beams. All temporary bracing for the shafts was designed by the contractor for a 5 ksf radial pressure distributed uniformly around the circumference of the shafts.

The intake shaft liners began as 24 ft 4 in. square and transitioned to 24 ft 4 in. diameter circles (see Figure 1). As the section approached a circular shape, the shaft liner thickness reduced from 4 ft 6 in. to 1 ft 6 in. where it is completely circular in shape. The outlet shafts have an inside diameter of 35 ft 0 in. and has a liner thickness of 1 ft 6 in. (see Figure 2). There is a great potential for heave of the clay-shale surrounding the shafts. Therefore, expansion joints with waterstops were provided at all shaft to above ground structure interfaces. The expansion joints are capable of surviving 6 ft of vertical movement. All piping located in the outlet shafts was provided with flexible connections at the ground level.

Tunnel

The tunnels are lined with a 1 ft 0 in. thick precast concrete liner composed of six segments that are 4 ft 0 in. in width (see Figure 3). The liner required concrete with a 28-day compressive strength of 6,000 psi. The Contractor was given the option of providing the final design of the liners. The Contractor proposed the use of a six-segment precast liner with tongue and groove radial (longitudinal) joints and a butt joint for the transverse joint (see Figure 4). The butt joints also had 3/4 in.-diam dowel pins that were for alignment purposes. The Contractor also believed that the dowel pins would hold the transverse joints closed during the "push" cycle of the TBM and would also help maintain circularity for the precast liner segments. The flat portion of the radial joint also serves as a well defined bearing area for transmission of the very high axial loads in the liner due to the radially applied 10 ksf swell pressure that the clay shale could exert on the liners. The design of the liners was actually controlled by the bearing of the liner segments on each other. This is because the actual contact area of the liner segments is less than the cross-sectional area of the liner. As it turns out, the assumed bearing area could be increased due to the neat cement grouting of the annular space between the liner and the excavated hole. This was discovered after coring some of the joints on the San Pedro Creek Tunnel.

There are some negative aspects of the particular joint design chosen by the Contractor. The "ears" of the joint are subject to breakage during handling and erection. Also, any rotation of the liners at the joint will cause the liners to bear on corners and could also cause breakage of portions of the liner at its joints. The pins could be difficult if not impossible to install when there are alignment and grade problems. All these items were discussed at great length with the contractor; however, the Contractor chose to continue with his decision since the joints had performed well for him on a previous project.

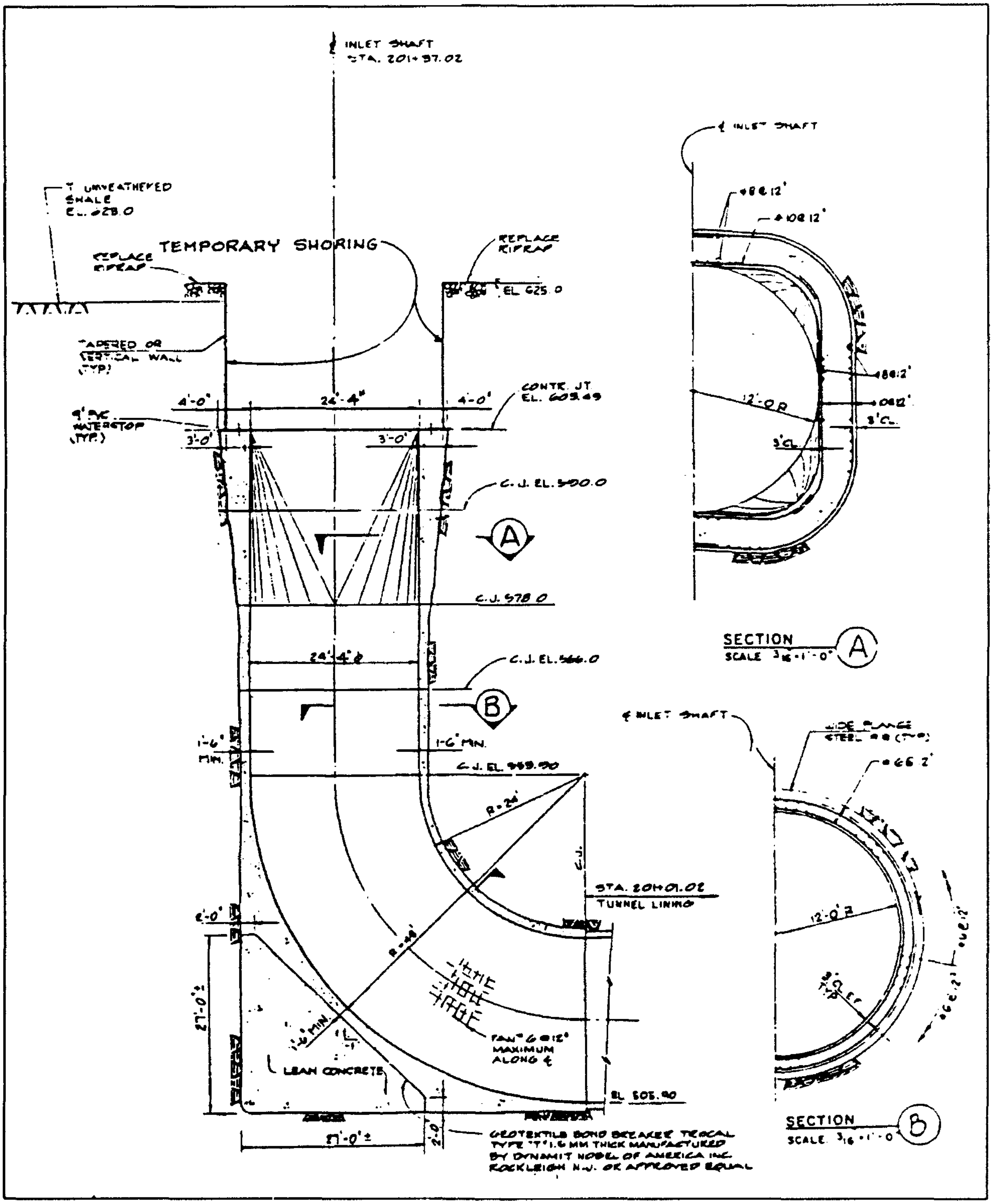

Figure 1. Inlet shaft sections

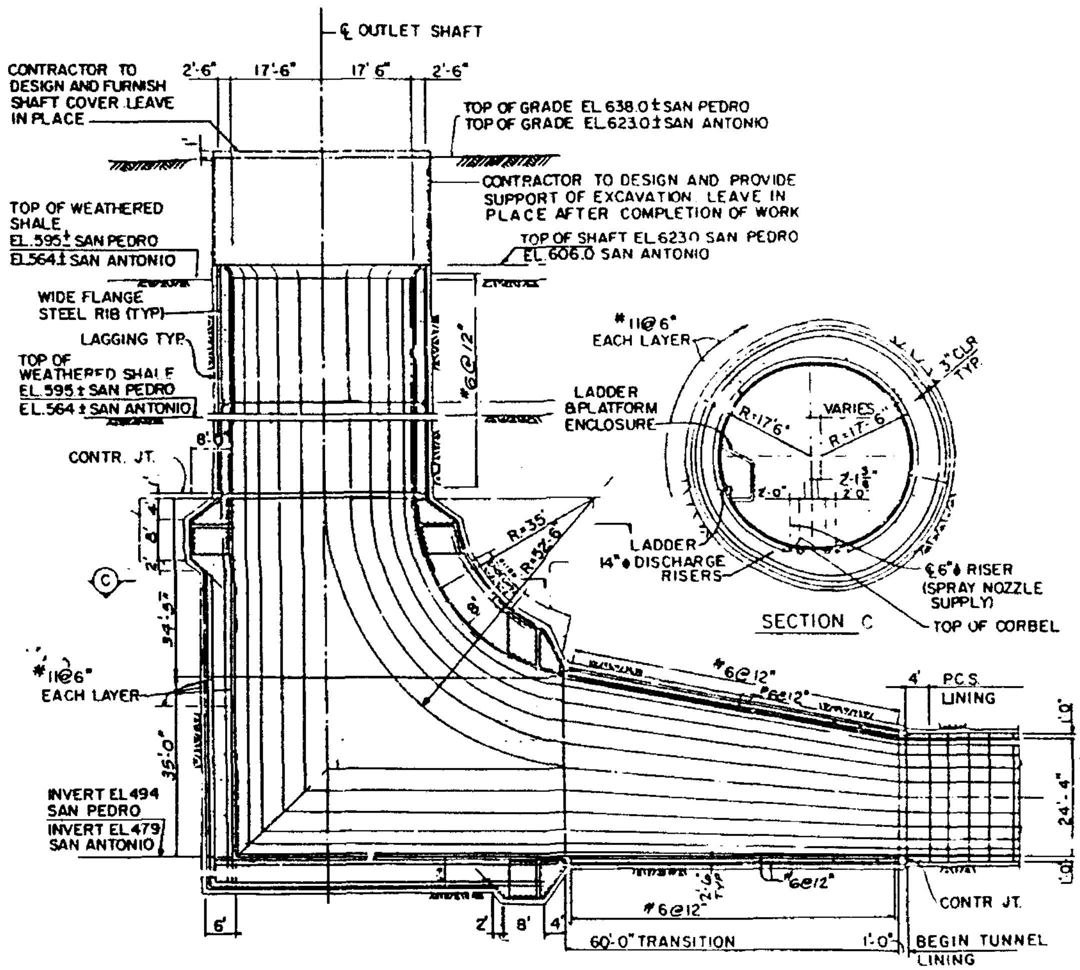

Figure 2. Outlet shaft sections

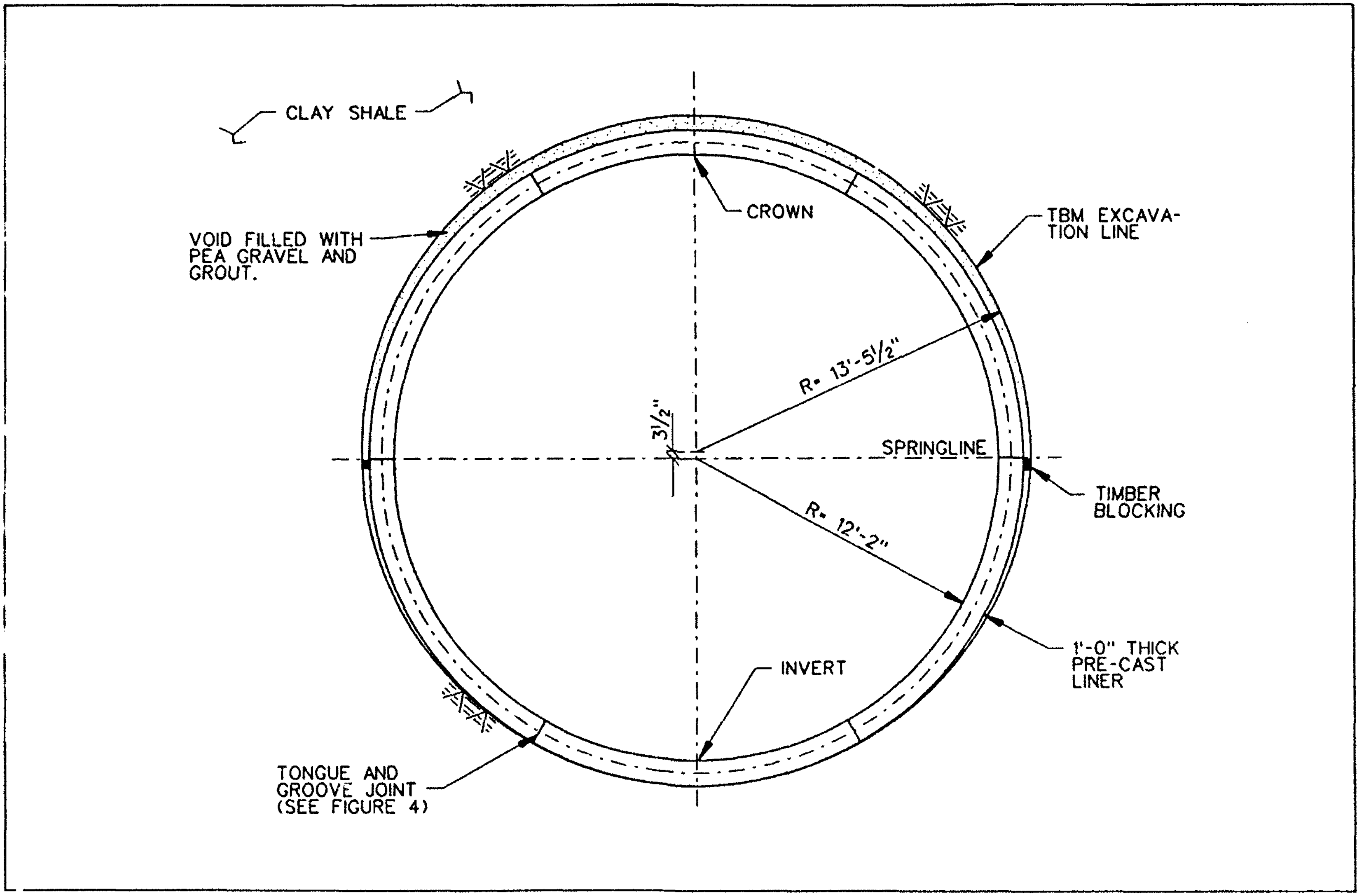

Figure 3. Typical tunnel section, ideal conditions

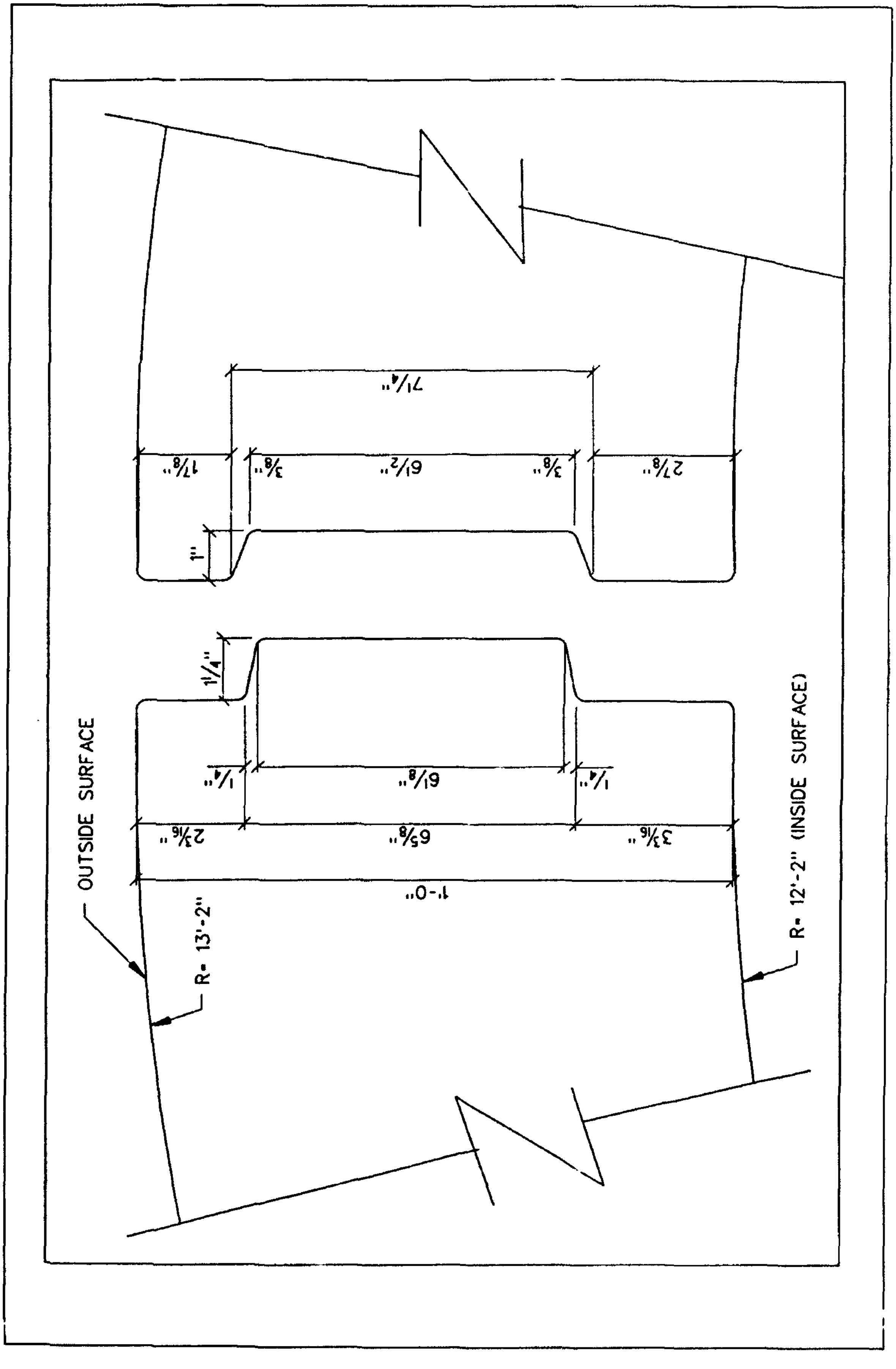

Figure 4. Tongue and groove joint detail

The general procedure of the precast liner installation was as follows:

- Excavate a 26 ft 11 in.-diam hole by the TBM.
- Three inches of pea gravel placed in the invert for invert segment (This was later modified to place the invert segment directly on the excavated surface as shown in Figure 3).
- Invert segment placed on pea gravel bed.
- Lower side segments installed with blocking at springline.
- Top side segments installed and held in place by TBM.
- Tapered crown segment installed.
- Pea gravel blown over completed ring of segments, covering as much of the ring as possible.
- The TBM excavates another 4 ft 0 in., and pushes forward to set another segmental ring.
- First stage grouting occurred after trailing gear of TBM passes (400 ft behind the TBM).
- Final grouting and coring program to ensure intrusion of grout to all voids.

Detailed Descriptions

San Pedro Creek Tunnel construction

The excavation of the inlet and outlet shafts for the San Pedro Creek Tunnel (SPCT) was as described earlier; however, the installation of the precast liner had some problems. All of the negative issues that were pointed out that could happen, did happen. Breakage at the radial tongue and groove joints was common place on the curved portions of the SPCT alignment. Unfortunately, the drilling of the SPCT began on a 1,273 ft radius curve. This coupled with the time it takes the TBM operator to become adjusted to the handling/operation of the machine made the first 600 ft of the SPCT very rough going. The transverse joints were also damaged due to the TBM "pushing off" the liners. When negotiating a curve, the TBM was not pushing off in a direction parallel with the longitudinal axis of the last segmental ring. Therefore, the loads on the transverse joints were not perpendicular to the surface of the joint. This caused portions of the joints to be broken and spalled off. The breakage was minimal on the straight portion of the alignment. The Contractor bolted steel plates across the crown joints as a precautionary measure. The plates were used to prevent the crown joint from opening up and causing a potential collapse of the liners if too much of the bearing surface had been damaged.

The dowels were also rendered useless. The TBM got off grade and alignment trying to negotiate the first curve. This caused the dowels not to line up with their holes. The lack of the use of the dowels does not affect the capacity of the liners to resist the applied swell pressure.

All the damaged portions of the liners were repaired by removing damaged/broken concrete and replacing with epoxy concrete. Areas where the TBM was off grade, the Contractor was allowed to offset the placement of the liner 1 in. in a vertical direction to get the bore back on grade. These 1 in. offsets were allowed by the hydraulic designers as it would not impact on the flow capacity of the tunnel.

After the Contractor had completed the second round of grouting, a coring program was implemented to ensure grout intrusion into the annular space between the liner and the excavated bore. Also, it was questionable if the grout could flow into some of the smaller crevices of the tongue and groove joint on the back side of the liners. The coring program provided positive evidence that the grout was filling the annular space and also the tight crevices of the tongue and groove joint. The joint, after grouting, has a positive bearing area of almost 9 in. instead of the assumed 6 in. Also, the liners were brought into intimate contact with the

surrounding clay shale mass by pressure grouting, thereby ensuring the stability of the segmental ring/excavation interface.

San Antonio River Tunnel construction

Excavation of the vertical portion of the San Antonio River Tunnel (SART) outlet shaft went without any problems as was described earlier. However, movement of the wall of the outlet shaft in the area of the bend (as the shaft goes from vertical to horizontal) occurred. This was caused by the presence of slickensided joints, along with sandy silt seams on bedding planes and the low strength of the rock. This area of the shaft required sixty 40 ft 0 in.-long rock anchors to stop the movement.

These same geologic problems caused fallouts ahead of the TBM during excavation of the tunnel. Excavation was very rough with frequent fallouts. The TBM was off grade and alignment and proving very difficult to control. After excavating approximately 30 ft, the TBM became lodged with falling blocks of clay shale. A cavity 15 ft high above the crown was created by the falling material. The fallout material jammed the TBM cutter head, preventing it from turning. The TBM was pulled back into the transition area of the shaft. A bulkhead was built across the SART portal. The Contractor drilled a bore hole from the ground surface into the crown of the fallout cavity. A 6-in.-diam steel casing was installed in the boring to backfill the cavity with lean concrete. The cavity held 768 cu yd of lean concrete which was placed over the fallout rubble. After the concrete was allowed to set, the bulkhead was removed and the TBM was used to excavate through the concrete. Excavation proceeded without any problems; however, as the TBM approached the outer limits of the concrete filled cavity, the ground began to rubble again but the contractor was able to continue, with difficulty.

The hole that was being excavated by the TBM was very irregular in shape due to the rubbling nature of the surrounding shale. Usually the excavated hole was irregular from springline and above with a cavern forming above the crown (in the form of a chimney). Auxiliary support was installed on the precast segmental rings starting with ring No. 10 (40 ft from the transition) to help keep the precast liner circular (see Figure 5). Auxiliary support was installed on rings No. 10 through No. 38, Nos. 63 and 64, and Nos. 70 through 80.

Damage to the crown segment of rings Nos. 30 to 33 was caused by rubble/concrete falling onto the crown segments. The loose material surrounding the liner combined with the voids, allowed bending moments to develop in the crown segment with a very small amount of axial force. The liners were designed to resist axial forces primarily and had a very low bending capacity. These crown segments will be repaired or replaced. There was other damage to the liner joints similar to the damage on the liners at the San Pedro Creek Tunnel. Damage was due to the TBM pushing off the liners and due to the liners being forced into place because of the non-circular position of the liners.

It was decided to stop the TBM operations before going under the schoolhouse property. Since one of the chimneys formed by the fallout had risen 40 ft above the crown of the tunnel crown, there was some concern that the fallout could rise high enough to reach the foundations of the structures above. The reason for the stop was to explore other avenues to complete the tunnel or to continue with rough going and liner damage and still be very uncertain about the adequacy of the tunnel constructed.

A decision was made to try and reinforce the shale above the crown of the tunnel by drilling a grid of belled piers down 1 to 2 ft above the crown of the tunnel. Eighteen rows of drilled piers were spaced at 8 ft along the longitudinal axis of the tunnel. The piers were spaced at 8 ft centers transverse to the axis of the tunnel with an alternating pattern of three and four piers per row. A stress field moves ahead of the tunnel excavation as the hole is opened up. The low strength of the clay shale, the sand seams, the slickened sides, and high

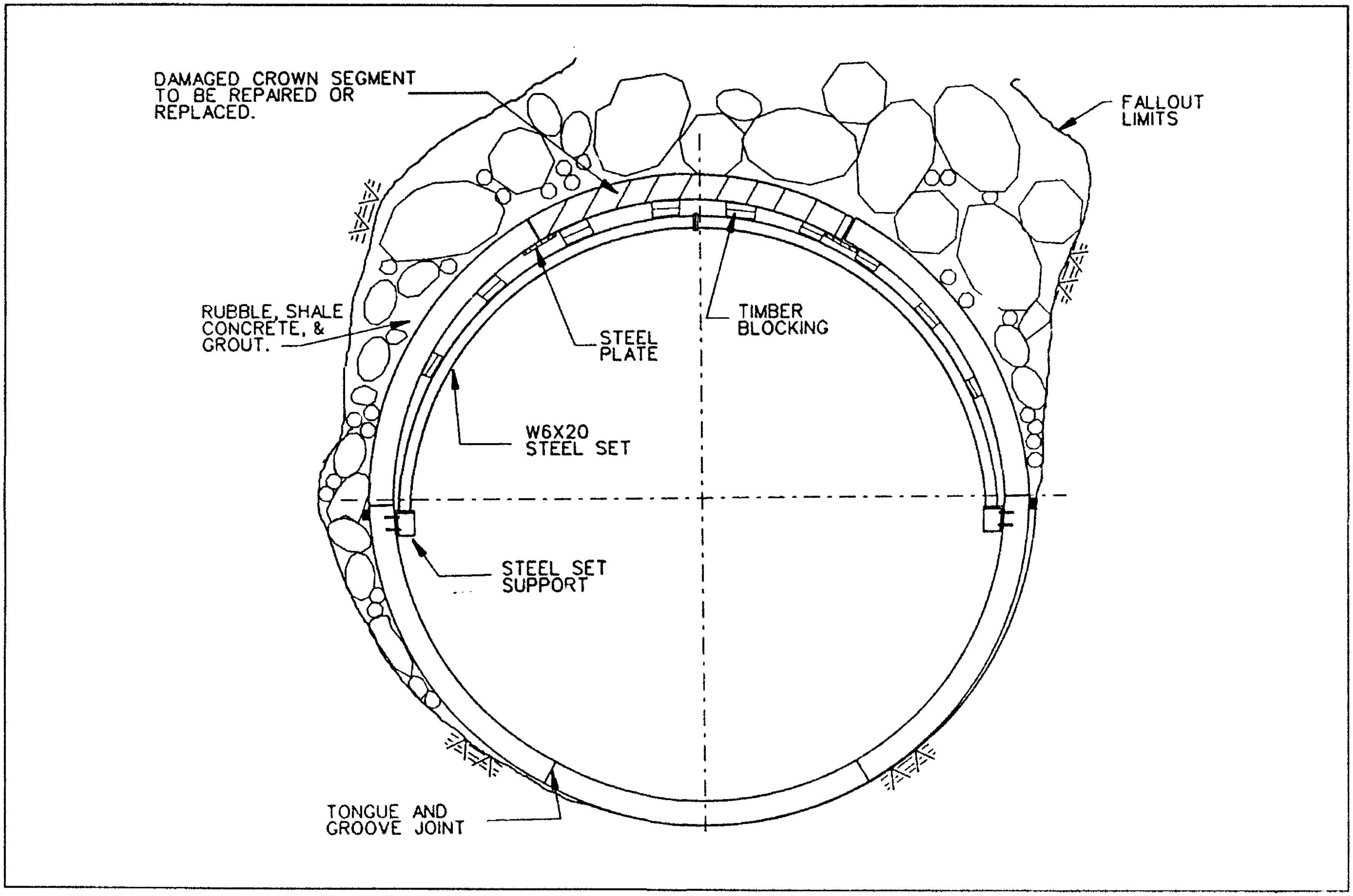

Figure 5. Typical tunnel section, nonideal conditions

angled faults combined with the stress field formed during excavation was the main reason for the fallouts that were occurring. It was thought that the drilled piers would hold the shale above the crown together and permit the TBM to excavate ahead of the stress field and allow time for the liners to be installed.

The drilled piers performed very well. The excavation under the piers went well with fallouts being contained. No auxiliary supports were required in this section of the tunnel (rings Nos. 39 to 69) except at rings Nos. 63 and 64 where the crown segments were cracked due to falling rubble onto the segments. However, as soon as the excavation moved out of the vicinity of the drilled piers, the Contractor had a mechanical problem with the TBM and was shut down for a period of time. The shale was unable to stand unsupported for this period of time and began falling on the cutter head jamming it. By now everyone involved including the Contractor was very discouraged. The Contractor informed the Corps that this condition seemed to be continuing and probably would continue until we were out of this particular material (2,000 ft). The TBM was now stuck after excavating approximately 400 ft. The last ring of segments installed was No. 80. The void above the TBM was filled with sand, and the TBM was prepared for an extended shutdown period.

The Contractor was asked to submit his design on the manner in which to proceed with the excavation. The Government was also in consultation with its consultants. A top heading consisting of W8X40 steel sets curved to a radius of approximately 16 ft and spaced at 4 ft 0 in.-centers was presented by the Contractor. The steel sets were founded on a berm about 3 ft above invert of the top heading. The ribs were supported on footings of sacked concrete and later founded on timber boards. Wooden blocking, timber lagging, and cribbing were to be used between the steel ribs and the excavated surface. The Contractor presented this in the form of a proposed modification to the contract. Provisions were also allowed for the use of spilings if the ground conditions dictated it. The Government agreed with the proposal with some reservations. The major reservation that the Government and its consultants had was with the use of timber lagging and blocking. The blocking would be difficult to use in areas of overbreak. Also, the timber did not have enough capacity to hold the shale between the steel sets if fallouts occurred. Essentially, the Contractor's plan was described as passive support of the excavation. An active support system would be needed based on what had transpired during the excavation of the SART outlet shaft and during the TBM excavation. The Government's concerns were discussed with the Contractor, but he felt that his plan was workable and adequate for its purpose.

This plan required the sinking of a 21 ft 0 in.-diam temporary shaft at approximately sta 23+60 and drilling the top heading downstream to the TBM at sta 14+00 to "free" it. Then the top heading excavation would return to the location of the temporary shaft and proceed upstream to sta 32+00. After repairs and modifications were made to the TBM, it would excavate the lower half of the tunnel from sta 14+00 to 32+00.

The Contractor proceeded with his plan for excavating the top heading. It became apparent immediately that spilings would be needed. The spiling consisted of No. 11 rebar and later 2-1/2 in.-O.D. pipe, 14 ft long. However, the spilings were not placed in grouted holes. This made the spiling practically useless in reinforcing the excavation.

There were at least two caverns that developed due to the fallout that started at the face and went up and forward. The caverns were supported with wooden cribbing stacked as high as 15 ft 0 in. above the steel sets, then the cavity was filled with concrete. Cribbing was also used to "block" the steel sets to the excavated surface of the shale where there were overbreaks. Shotcrete was then applied to lagging (cribbing) and the steel ribs after they were installed. Shotcrete was applied by personnel standing on the floor of the top heading resulting in considerable rebound. Shooting shotcrete standing on the floor

placed the nozzle too far from the clay shale surface and makes it difficult to fill voids behind and between the lagging and blocking.

This was the description of the top heading operation when on July 30, 1990, the top heading excavation collapsed between ribs Nos 35 and 49 at the face. The collapse was preceded by cracks in the shotcrete, chunks of shotcrete falling from the crown, and bits of shale falling from behind the shotcrete. The Resident Engineer was in the top heading during this period. He noticed the shale "working" and yelled to the Contractor's supervisor to stop all work and remove the workers from the face area just before the shotcrete started crumbling and falling on a large scale. Within a few minutes the ribs began to fail and depress inward from the crown.

The Government has taken the position that the collapse was a direct result of the Contractor's construction methods. The steel sets were not blocked to the excavated surface with shotcrete and in some cases shotcrete could not be placed on the excavated surface because of all the timber cribbing/lagging. This allowed the shale to loosen and fallouts to occur. The Contractor is not in agreement with this; however, the Contractor was informed that he had not been relieved of any of his responsibilities under the mod for constructing the top heading. This meant that the remine of the failed section (ribs Nos. 35 to 49) was at his expense. The Contractor has placed the Government on notice that a claim will be filed on this issue.

The Contractor remined the failed section using the techniques that the Government had preferred. The shale was excavated, shotcrete was applied, and then the steel rib was installed and blocked to the shotcrete with shotcrete placed between the flange of the steel set and the previously applied shotcrete. A final layer of shotcrete covering the entire rib was applied later. Using this method, the Contractor has been able to control all unfavorable conditions encountered during the remine section and the remainder of the excavation toward the TBM. The Contractor had an average rate of advance of approximately 12 ft per day. The top heading excavation is projected to be completed by July 1991.

Conclusion

The construction of the SART in particular has been filled with excitement, confusion, disappointment, despair, and elation again. The borings that were obtained in this area did not provide enough information to predict the blocky nature of the clay shale to its full extent. The Geologists knew that the material was somewhat softer than that in the SPC Tunnel. They were also aware of the sand seams and the bentonite seam about 15 ft above the crown of the tunnel. The recovery of all the bore holes was almost 100 percent. The extent of the overbreakage and chimneying was not foreseen. Perhaps the possible problems that could have happened were lost on another problem that was foreseen with the clay shale material, and that was its expansive nature. However, it should be pointed out that a shielded TBM was required by the contract specifications because there was a potential for rock fallouts, but the magnitude of the blockiness was not foreseen.

It is evident that an active support system was required for the excavation of this material. The face of the excavation should have been supported at all times. The problem with the modified hardrock TBM was that the cutter head had to be pulled away from the face to start the head turning. This was where all the problems started while excavating with the TBM. Also, final support should be installed as soon as possible after the excavation is opened. That means the distance from the face to the installation of the liners should be as short as possible.

The liners should have been pressure grouted sooner than 400 ft behind the TBM. The clay shale needed full support as soon as possible. Until the liners were contact grouted, the clay shale was allowed to loosen, thereby creating voids and fractures. It should be pointed out that not only does the liner support

the shale, but the shale offers support to the liners so that the excavation is supported by axial compression in the liners, not bending.

Finally, the use of an active support system should always be pursued for tunnel support. This allows the rock to support itself by arching action. The corresponding loads to any external support will be reduced. Also, the chances of controlling any unfavorable events during tunnel driving will be more easily controlled. But, the data obtained from the exploratory drilling must be adequately and closely scrutinized. Because in my limited experience in tunnel construction, I have learned that "if anything can go wrong, it will go wrong."

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Bassett Creek Tunnel Flood Control Project

by
Thomas B. Sully[1]

Abstract

The St. Paul District, US Army Corps of Engineers, has begun a flood control project in Hennepin County, MN, on Bassett Creek. Bassett Creek is a tributary of the Mississippi River, and its watershed is entirely within the Minneapolis metropolitan area. The watershed is in a 70 percent state of urban development and is therefore susceptible to flash flooding.

The project is divided into four stages with the lower watershed consisting of one major feature - a new tunnel to replace an existing, deteriorating tunnel.

This paper briefly describes the overall design and construction techniques used in this reach of the flood control project.

Introduction

The Bassett Creek Flood Control Project is a four stage, $30 million initiative of the St. Paul District of the Corps of Engineers involving the diversion of Bassett Creek. The tunnel feature is a cooperative venture with the city of Minneapolis and the Minnesota Department of Transportation (MnDOT). Stage 3 (Third Avenue Tunnel) consists of approximately 1,400 ft of deep tunnel about 75 ft underground. It connects at its downstream end to a MnDOT tunnel which was constructed between 1979 and 1981. The tunnel connects at its upstream end to a double box culvert constructed under another stage of this project.

The tunnel section is a 11 ft 6 in. wide by 15 ft high, reinforced concrete cathedral shape and was excavated in the geologic formation known as St. Peter Sandstone. Also, part of this stage is a drop structure which is a 6-ft-diam round concrete pipe that connects to the main tunnel with a wye. This round pipe conveys runoff water to the tunnel from a nearby drainage area for Interstate 394.

Design

The existing tunnel is a box culvert configuration built in 1900. The existing tunnel was built in sections to enclose a channel between existing street crossings. The sections are of various diameters and materials. There is extensive deterioration of the concrete and its reinforcement. The tunnel has the potential for becoming blocked or collapsing which would cause extensive flooding upstream. Further, the tunnel would not have sufficient capacity for discharges from the proposed project.

The new tunnel is located completely within the St. Peter Sandstone. This sandstone can generally be characterized as poorly cemented, fine grained, homogeneous, and porous. The lack of cementation makes excavation very

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easy but results in overbreak at the tunnel crown resulting in an arch shaped opening.

The tunnel configuration was a cathedral shape approximately 11 ft 6 in. wide by 15 ft high (see Figure 1). This configuration relates to an equivalent for a 13-ft-diam round pipe. The cathedral shape allowed the tunnel to react in such a way that very small flexural stresses were realized. The hope was to eliminate the need for reinforcing steel. This turned out to be impossible however because during the design event, the hydraulic grade line is 36 ft above the invert and a soil-structure interaction analysis indicated that although the sandstone surrounding the tunnel would resist most of the outward pressure, the concrete shell would still experience tensile stresses exceeding the allowable amount for an unreinforced section.

A chemical analysis of the groundwater indicated the presence of sulfates. Because of the presence of sulfates, all reinforcing bars were epoxy coated. Type 2 concrete was also used for sulfate resistance and to control heat generated during the hydration process and thus associated cracking. Controlling the rate of hydration heat production was of critical importance. The concrete cured bounded on one side by formwork and air, and on the other side by wet sandstone, which made an effective heat sink. Large and damaging temperature gradients can result in this kind of situation if hydration heat is produced at too high a rate. The minimal cracking that did occur was repaired with epoxy injection so that sandstone outside the shell would not be transported into the tunnel forming a void on the outside.

Construction

The construction sequence was divided into five major phases:

- Phase 1 - Start Up, Excavate Access Shaft
- Phase 2 - Major Mining of Tunnel
- Phase 3 - Construction of the Wye
- Phase 4 - Tunnel Lining
- Phase 5 - Grouting and Crack Sealing

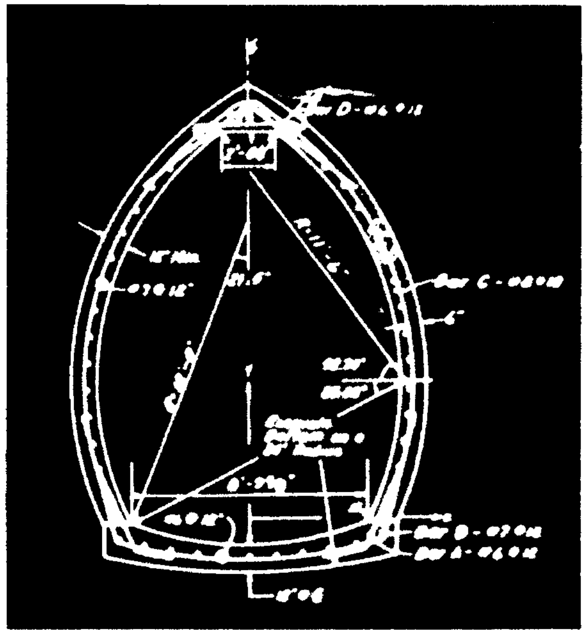

Figure 1. Cathedral arch tunnel

Phase 1

The initial work required construction of an access shaft from the ground surface down to the tunnel elevation at the upstream and downstream end of the project. The shaft was located close to the center of the alignment rather than at one end. This expedited construction in two ways. First, it allowed work to progress in two directions simultaneously. Secondly, it cut the average material transport distance in half. To allow mining below the watertable, it was necessary to install deep dewatering wells to control seepage. The entire 1,400-ft alignment was dewatered with only four wells. Once the access shaft was completed, the tunnel mining could proceed.

Phase 2

The first phase of the tunnel mining operation progressed from the access shaft downstream to connect with the wye at the existing MnDOT tunnel (see Figure 2). Once the wye was reached, and tied into, the concrete lining operation was initiated progressing from the wye back upstream. While this was going on,

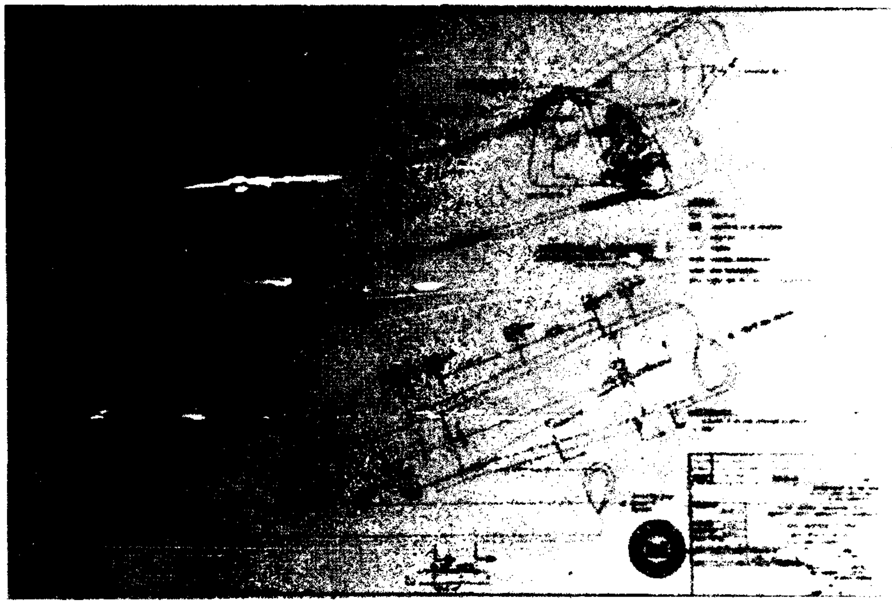

Figure 2. Wye connection to existing tunnel

the tunnel was mined from the access shaft upstream to the location of the Drop Structure which would be constructed in the next stage of the project. Mining was accomplished with high pressure (2,000 psi) hand held water lances which were used to outline the general tunnel shape and cut the sandstone intopieces (see Figure 3). Hauling of excavated material was done with a CAT 933 front-end loader which traveled back and forth between the excavation face and the access shaft (see Figure 4). Temporary support of the excavated tunnel was accomplished by first spraying a solution of sodium silicate on the freshly excavated surfaces to prevent unraveling. Then rock bolts were epoxy grouted into the tunnel crown which supported wire mesh (cyclone fence) (see Figures 5 and 6). Shotcrete was also applied in a few areas.

Phase 3

The wye at the MnDot tunnel needed major modification because it had been sized for a 10-ft equivalent diameter connection rather than the 13 ft equivalent diameter that was actually constructed. This was because in the years between the construction of the MnDOT tunnel and the Corps' Bassett Creek Stage 3 tunnel, the area hydrology changed due to urbanization and the design event increased from 600 cu ft/sec to 1,000 cu ft/sec or more.

Phase 4

As mentioned above, the location of the access shaft made it possible for the lining operation in the downstream portion of the tunnel to proceed concurrently with the mining operation in the upstream portion. Lining proceeded as follows:

The reinforcing for the invert was set and the invert was poured. This provided a solid working surface. The reinforcing for the side walls and crown was placed. Twenty-foot long steel form sections supported by a rubber-tired carriage were then rolled into place (see Figure 7). The forms were hinged at the crown so that the side walls could be retracted. The carriage also had screw jacks to raise and lower the forms. In this way, the forms could be pulled off cured concrete, advance to the next pour in a retracted position, and then be jacked back into place. Then, concrete was pumped into the forms from the surface through pipes. Slump was specified 7 in. at the discharge end of the pipe line. Consolidation was accomplished largely though the use of

Figure 3. Setting tunnel outline template prior to water lance mining

Figure 4. Excavation with front-end loader

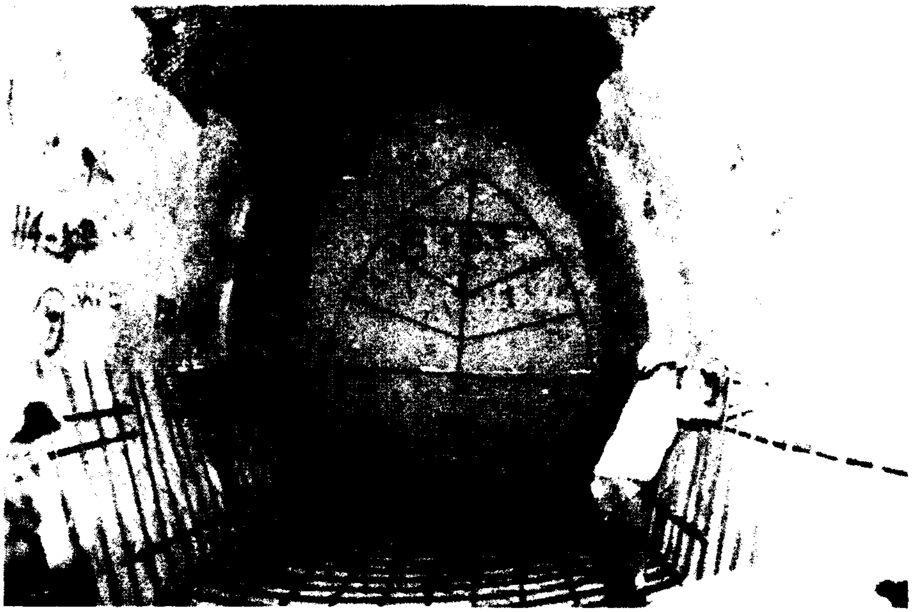

Figure 5. Tunnel crown support, rack belts

Figure 6. Tunnel crown support, cyclone fence

Figure 7. Tunnel sidewall forms

external form vibrators. The concrete attained sufficient strength to allow form pulling in 12 hr.

Phase 5

After the liner was in place, holes were drilled into the crown at 10 ft-intervals. Sand grout was then pumped through the front to the back of the liner to fill any possible voids.

Crack sealing was done by injecting every crack of width more than 0.01 ft with epoxy crack sealant. Due to excellent heat control and fine workmanship in the concrete placement, the number of cracks was small. Cracks usually occurred at construction joints, rarely elsewhere.

Conclusion

The project was completed on schedule with only one major construction claim due to an unforeseen site condition in one area of the tunnel.

In-Structure Shock Prediction for Buried Structures

by
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Abstract

A survey was conducted of available dynamic test data and current in-structure shock analysis techniques. Simplified semi-empirical, single-degree-of-freedom, and two-dimensional (2-D) finite-element, in-structure shock analyses of buried reinforced concrete structures were performed and compared to test data. An evaluation of the analysis techniques was completed and a technique was selected for further development as an in-structure shock analysis tool.

The technique selected for further development, based on this evaluation, is the explicit 2-D finite-element program ISSV3. Extensive comparisons of the results of the in-structure shock analysis of the test configurations, using the available methods, show ISSV3 to be fast, accurate, and easy to use.

Introduction

The analysis of buried structures designed to resist blast loading is a continuing problem in the area of engineering mechanics. The complex structural dynamics of the problem, coupled with the uncertainties of soil structure interaction, make this a most difficult challenge. Even if the structure itself is designed to survive the expected dynamic loads, the shock environment inside may be severe enough to harm personnel or equipment. A thorough understanding of the expected shock environment is therefore needed to design ways to reduce this damage. The present simplified methods for the analysis of this type of problem are only directly applicable to very simple configurations and are thought to yield very conservative results for more complex structures. As a result, effort is now being expended to mitigate shock levels which may be unrealistically high. This program of study was prompted by the hope that a relatively simple, fast, and accurate analysis procedure could be found or developed, which will realistically estimate in-structure shock for complex buried reinforced concrete structures. This capability could then be used to improve the total design quality of such facilities. The objective of this study was therefore to conduct and evaluate in-structure shock calculations for buried reinforced concrete structures under dynamic loads from buried conventional explosives. The study included simplified semi-empirical, single-degree-of-freedom (SDOF), and two-dimensional (2-D) finite-element, in-structure shock analyses of buried reinforced concrete structures and comparison of the results to test data.

Shock Spectra

The shock environment inside of a dynamically loaded structure is a complex phenomenon. The amplitude, shape, and duration of the acceleration, velocity, and deflection-time

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histories greatly influence the response of internal systems that may be damaged by the shock environment. To simplify the design and analysis of internal systems against in-structure shock, a shock spectrum is generated to characterize the shock environment inside the structure. A shock spectrum is a plot of the maximum response of an SDOF system in which the base is driven by the in-structure shock environment. As can be seen in Figure 1, the response of an SDOF system is dependent on the natural frequency of the system determined by the mass and spring stiffness, the damping ratio, and the motion driving the system. Given the motion and damping parameters, the maximum response is calculated for a range of natural frequencies and a shock spectra generated. For many shock spectra the natural frequency of the SDOF system is on the abscissa and the ordinate presents the maximum response such as peak spectral acceleration, pseudo-velocity, or relative displacement. Peak spectral acceleration and pseudo-velocity are terms that closely approximate the actual acceleration and velocity of the SDOF system, and the relative displacement is the actual value. Thus, for a given internal system such as a piece of equipment, all that is needed to define its maximum response to a shock environment is the natural frequency of the piece of equipment and the spectrum of the environment. Several examples of shock spectra are presented at the end of this paper. Note that on these spectra the acceleration,

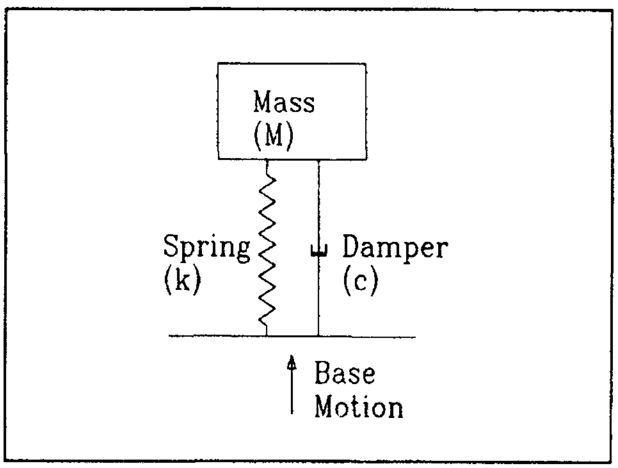

Figure 1. Single-Degree-of-Freedom System

velocity, and deflection all appear on the same plot. Shock spectra are discussed in detail by Clough and Penzien (1974), Biggs (1964), and Newmark and Rosenblueth (1971). These are good references for anyone interested in exploring the subject to greater depth.

CONWEB Test Series

The US Army Engineer Waterways Experiment Station conducted the CONWEB tests, a series of four backfill effects tests from March through May 1989. The test procedures are covered extensively by Hayes (1989). In the course of this test series, a large amount of structural acceleration data was recorded. These data, along with records of free-field acceleration, free-field soil stress, and interface stress, present a golden opportunity for the evaluation of in-structure shock calculation techniques. For this reason, this test series was selected as the primary focus of the in-structure shock analyses conducted. In the course of this study all four of these tests were examined; however, this paper will focus on the analysis of the second CONWEB test which is CONWEB 2.

CONWEB 2 was conducted in a clay backfill with a relatively stiff test wall. This clay was characterized by a low shear strength and a low seismic velocity. Figure 2 shows the test structure and backfill properties for the CONWEB 2 test. The source of the localized dynamic loads in the test was a cylindrical explosive charge. This charge contained 15.4 lb of C4 explosives in a closed steel case 27 in. long with an inside diameter of 3.548 in. and a thickness of 0.166 in. The charge was oriented horizontally in CONWEB 2.

The test article consisted of a reusable reaction structure supporting the test wall. The test wall was bolted to the heavily reinforced concrete reaction structure, forming a relatively rigid joint. The test wall for CONWEB 2 was a one-way slab with a compressive and tensile steel ratio of 0.0043 and a length-to-thickness ratio of 5. The concrete compressive strength was 6,398 psi and the reinforcing yield strength was 6,7424 psi.

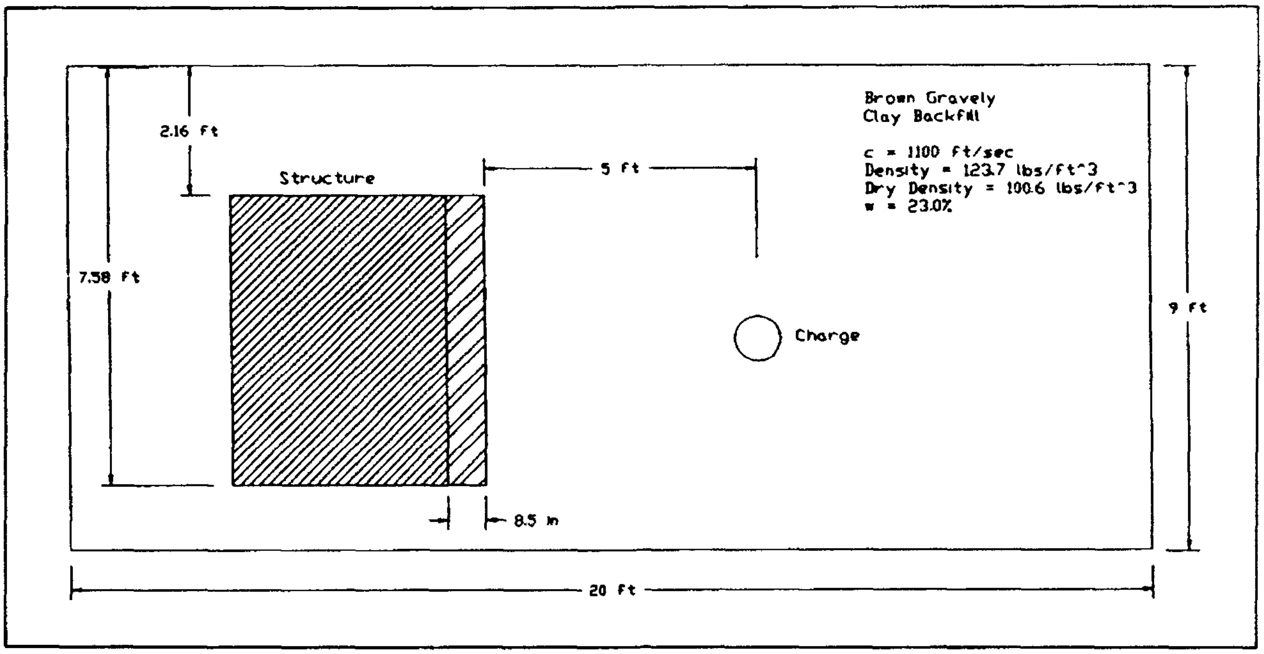

Figure 2. Section through CONWEB 2 test bed showing backfill properties

TM 5-855-1 Technique

A current method for the calculation of in-structure shock is the semi-empirical procedure in the Army Technical Manual (TM) 5-855-1 (Headquarters, Department of the Army 1986). The procedure begins with the calculation of the average free-field accelerations, velocities, and deflections. Correction factors are then applied to convert these free-field values to in-structure shock values. Further empirical factors are applied to create a shock spectra. Complete coverage of this method can be found in TM 5-855-1 (Headquarters, Department of the Army 1986).

The most important parameters controlling the free-field motion are the source of the ground shock, the mechanical properties of the free-field, and the range (R) to the point of interest. The source of the ground shock is determined by the size (weight) and depth of burial of the bomb. In order to simplify the analysis, the weight of the explosive is converted to an equivalent TNT weight (W) using equivalency factors. A ground shock coupling factor (f) is used to account for depth of burial. At the present time, the TM 5-855-1 procedure for the calculation of free-field motions contains only two explicit variables for the characterization of the free-field. These variables are the seismic velocity (c) and the attenuation coefficient (n).

In the calculation of internal motions for a structure buried in the free-field there are two ranges of interest, R_1 and R_2, the distance in feet from the front and the back of the structure, respectively. These two ranges, along with the parameters discussed above, are all that is required for the calculation of the average acceleration across the structure. Equations found in TM 5-855-1 are used to calculate the average acceleration, velocity, and deflection across the structure. In the case of CONWEB 2, an explosive weight (W) of 19.71 lb TNT, a coupling factor (f) of 1.0, an attenuation coefficient (n) of 2.25, a seismic velocity (c) of 1,000 ft/sec, and an R_1 and R_2 of 5.0 and 9.717 ft, respectively, were used in this calculation. A shape dependent reduction factor of 0.48 was applied to these results to give the in-structure shock motions. These motions were then converted to shock spectra values by multiplying the acceleration, velocity, and deflection by 1.6, 1.5, and 1.2, respectively.

The semi-empirical calculational procedure in TM 5-855-1 gives reasonable estimates of the in-structure shock responses of CONWEB 2. Figures 3 and 4 show the results of this calculation compared to floor motion test data. This procedure also produced generally good results for the other tests calculated. The greatest weakness of this procedure is that it was developed for simple box-like structures. The assumptions that must be made to apply this procedure to more complicated structures have not been thoroughly evaluated. Great caution should be used in the application of such an empirically derived method to situations outside of those actually tested.

SDOF Technique

The SDOF analysis method consists of the reduction of the problem to a simple spring, mass, and damper system. Application of this type of analysis is widely covered by Hayes (1989) and Biggs (1964). In an SDOF model, the spring is a resistance element which models the static resistance of the structural element of interest. The mass and damper are selected so that the resulting system will have the same frequency and damping characteristics as the prototype structure.

The SDOF analysis procedure was used to model the CONWEB tests in two ways, the wall facing the bomb was analyzed as one SDOF system, and the horizontal rigid-body motion of the entire structure was modeled in a second decoupled SDOF analysis. Output from the wall analysis was compared to data from gages on the wall, and the rigid-body analysis results were compared to data from internal gages on the floor. This ignores the contribution of the rigid-body motion to the response of the wall. Since the peak wall response occurs well before the peak rigid-body response, neglecting of the rigid-body contribution is thought to be a reasonable assumption.

Front wall SDOF analysis

The front wall SDOF analysis was carried out using the Corps of Engineers PC-based Wall Analysis Code (WAC)(Jones, Hetherington, and Coltharp 1988). WAC uses the procedures in TM 5-855-1 to develop the load mass factors for a given wall. Multilinear resistance functions are computed based on yield-line theory.

The wall loading used was a modification of the free-field stresses calculated by the procedures in TM 5-855-1 (Headquarters, Department of the Army 1986). The soil properties used were the same as those assumed in the TM 5-855-1 procedure discussed above.

Peak deflections, velocities, and accelerations resulting from the wall SDOF analysis were modified to create shock spectra values. These results can be seen in Figure 5 and compare well with the average measured front wall motions for CONWEB 2. However, the results of the wall SDOF analyses were relatively inconsistent for the other tests examined, though, it is thought too difficulty in developing reasonable loads to apply to the SDOF model. The loads used here were developed using the procedures in TM 5-855-1 (Headquarters, Department of the Army 1986). It is felt that these procedures do not adequately model structure-media interaction (SMI). There is current research at WES to address these deficiencies in SMI modeling.

Rigid-body SDOF analysis

A simple approximation of the horizontal shock environment on the floor of a structure is the overall rigid-body motion of the entire structure. Rigid-body motion was analyzed by reducing the problem to an SDOF system, with the total mass of the structure concentrated to a single point, and with the soil behind the structure acting as a simple linear spring. The load used was the same as that calculated for the wall SDOF above. The soil-spring constant was calculated using a procedure presented by Whitman and Richart (1967) for dynamically loaded foundations. The resulting soil-spring constant was 1.281×10^6 lb/in., and the total mass of the structure was calculated as 84.48 lb-sec^2/in. Frictional forces that exist at the top and bottom of the structure are neglected in this

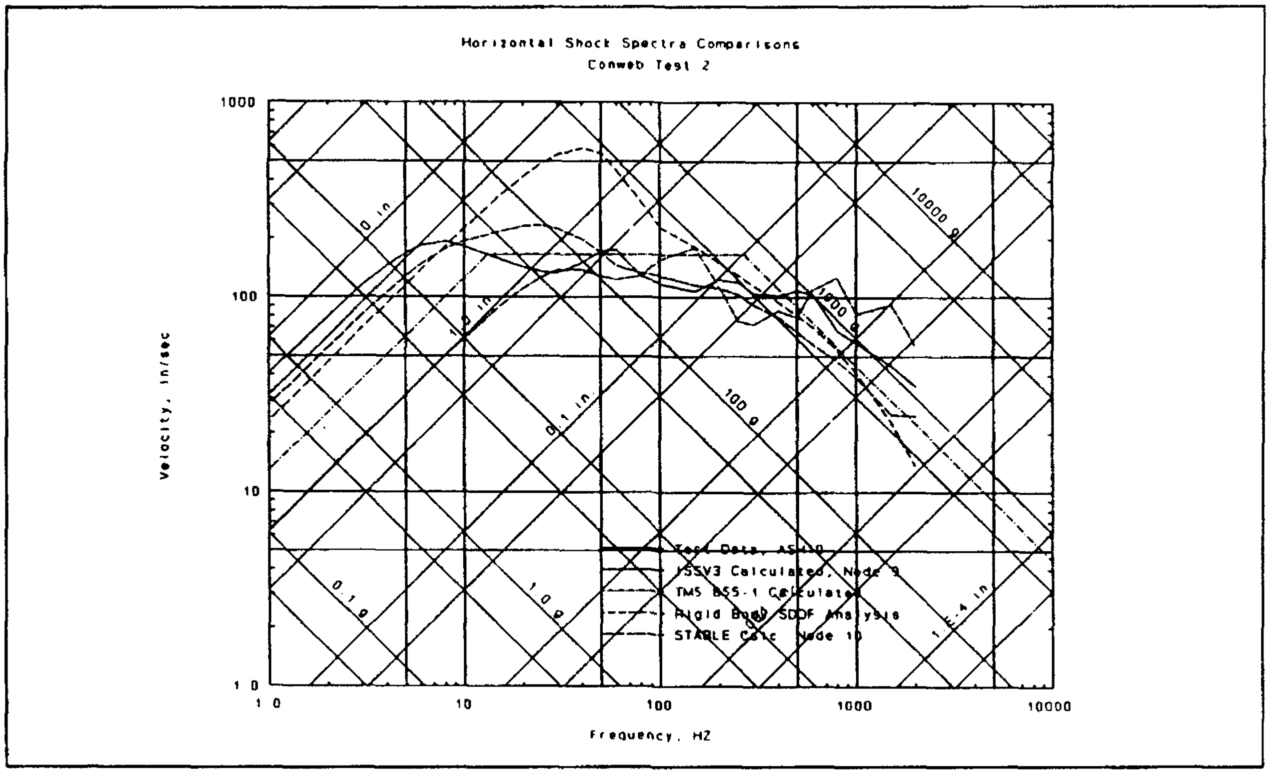

Figure 3. CONWEB 2, mid-floor horizontal shock spectra, ISSV3 Node 9, comparison of analysis techniques

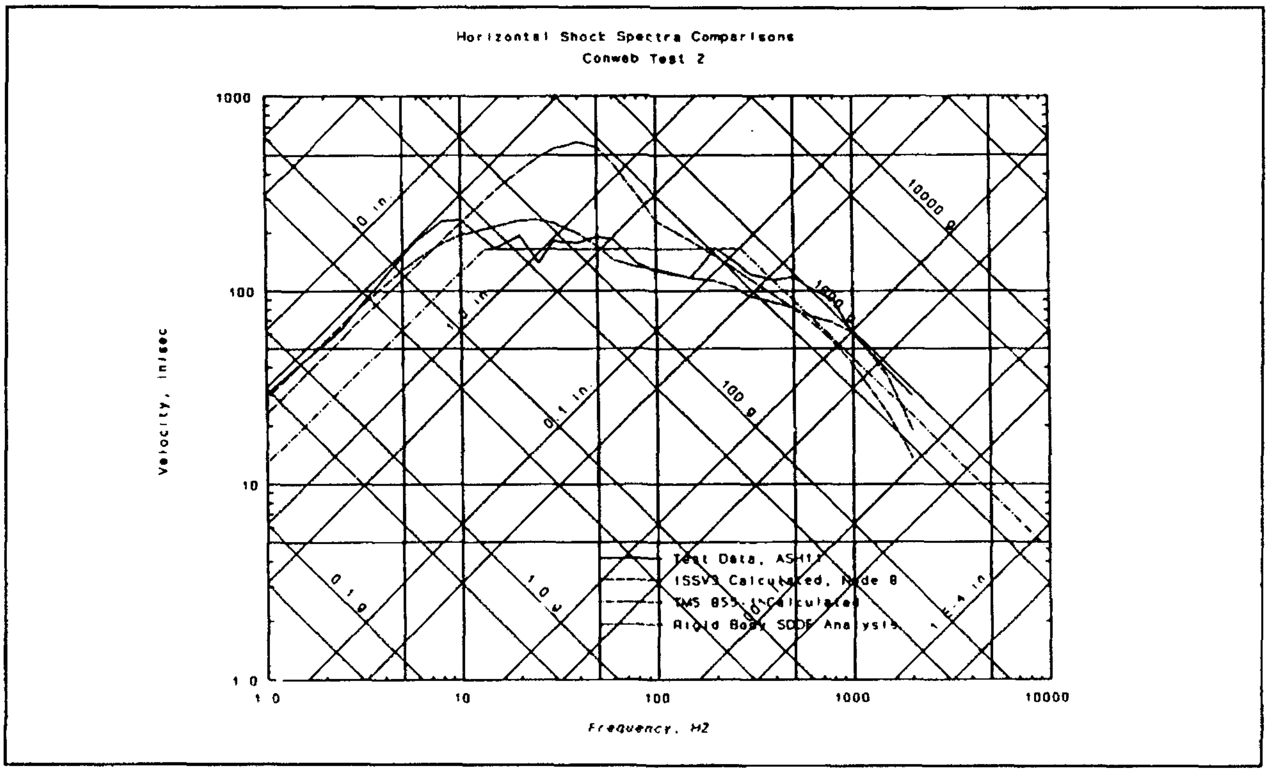

Figure 4. CONWEB 2, Floor horizontal shock spectra, ISSV3 Node 8, comparison of analysis techniques

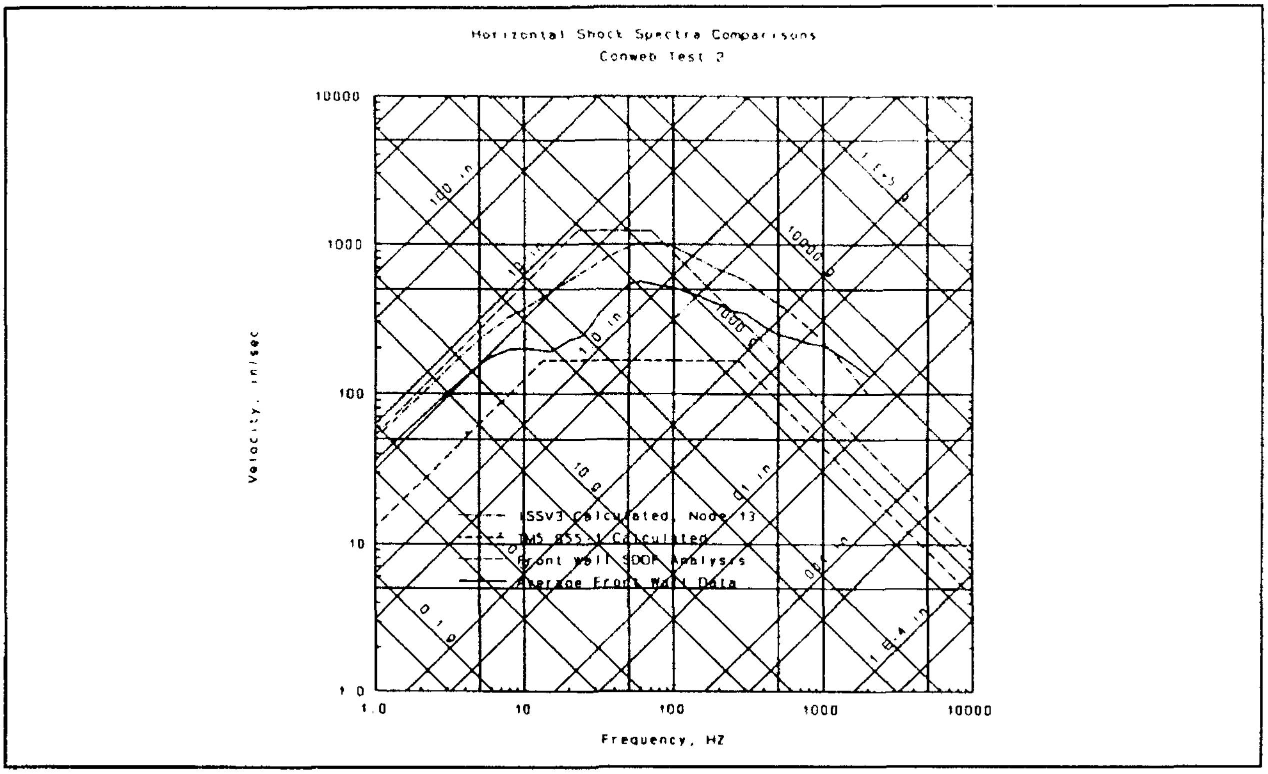

Figure 5. CONWEB 2, mid-wall horizontal shock spectra, ISSV3 Node 13 comparison of analysis techniques

analysis. A relatively low damping value of one percent of critical was included for stability and does not affect the first peak response.

Given the above mass, soil-spring constant, damping coefficient, and the load calculated earlier, the rigid-body SDOF analysis was completed using a computerized version of the procedures developed by Biggs (1964). Velocity-time histories up to the time of maximum positive deflection were input into the same shock-spectra generation program used for the test data.

As can be seen in Figures 3 and 4, the rigid-body SDOF analysis gave reasonable predictions of peak deflections and accelerations CONWEB 2. Peak velocities were overpredicted. The same trends existed in the other tests examined.

STABLE Finite Element Technique

STABLE is an implicit finite-element program for the dynamic analysis of frames subjected to blast and ground shock loadings (Campbell and Bryant 1989; Bryant and Campbell 1989; Campbell, Smith, and Flathau 1989). This program is in the public domain and was written by JAYCOR, Vicksburg, MS, for the US Army Engineer District, Omaha. The evaluation of STABLE as an in-structure shock analysis technique focused on the analysis of the CONWEB 2 test. The analyses, which were conducted on the CONWEB 2 test, included a coarse-grid analysis, a fine-grid analysis, and a fine-grid analysis with soil springs replacing the SMI at the back of the structure.

For each of these analyses, a 2-D finite-element grid was generated to model CONWEB 2.

The model consisted of a 1-ft-thick slice taken through the centerline of the structure. All major reinforcements in the structure are in this plane, and the effects of all out-of-plane reinforcement was neglected. Equivalent steel sections were used to model the concrete cross sections. Those interested in the details of these analyses should look to Dove (1991).

The evaluation of STABLE as an in-structure shock analysis technique led to the rejection of this program from further consideration. This rejection is not based on a problem of high frequency noise in the acceleration records that plagued this analysis. Favorable comparisons of the computational results to CONWEB 2 test data indicated that the program is capable of handling this type of application. Modification of the program to eliminate the problem of high frequency noise in the acceleration records is quite possible. The basis for the rejection of the application of STABLE to in-structure analysis is excessive run times on the target computer. The fine grid analysis required approximately 16 hr to complete on a 20 MHz, 386 personal computer.

Excessive run times for STABLE are directly related to its formulation as an implicit finite-element analysis program. The implicit formulation refers to the solution method used to solve the equations of motion as the problem proceeds through time. Using a large time step is the main advantage of such a formulation; however, with each time step, the solution of the equations of motion requires a relatively large amount of calculational effort and, hence, computer time. Unfortunately, the application of STABLE to in-structure shock analysis requires a small time step and a large number of iterations to capture the high frequency response of the structure and to keep the SMI calculation from going unstable. As a result, there is a combination of the worst of both worlds; that is, the requirement for many short time steps that require a large amount of computer time for each calculation. Modifications are possible that could increase the computational efficiency of STABLE, but it is felt that such modifications could not overcome such a fundamental limitation.

ISSV3 Finite Element Technique

With the rejection of STABLE, a new candidate program was selected for development as an in-structure shock analysis tool. The program, ISSV3, is an explicit type finite-element analysis program in which the time step must be small to assure stability, but the computational effort is smaller at each step or iteration. As will be seen below, the result is a program with orders-of-magnitude shorter run times.

ISSV3 was developed by Walker, Drake, Boyt, and Slawson (1990) of Applied Research Associates, Inc., Southern Division. The work was conducted under WES contract DACA39-90-0041 for Dr. J. P. Balsara and under supervision of the author of this paper.

As discussed above, CONWEB 2 was a test with the clay backfill material. The test wall was relatively stiff, with an L/d ratio of 5. A 2-D finite-element grid for the ISSV3 in-structure shock analysis of CONWEB 2 was generated in much the same way as in the STABLE analysis. In this case the program calculated all loads given the position and weight of the explosive and the soil properties. Section properties input into the ISSV3 program required some preanalysis. The ultimate moment and thrust capacities were selected from a calculated moment-thrust diagram for each section.

An example of the results from the CONWEB 2 analysis is shown in Figure 6. An examination of this acceleration record shows that the calculated time of arrival (t_a) of the load at the face of the structure lags the actual value by about 4 msec. This is due to the arbitrary nature of the calculated zero time. The shape and magnitude of the calculated acceleration record compares very favorably with the test data. On the whole, the calculated horizontal in-structure shock response compares very favorably with the test data

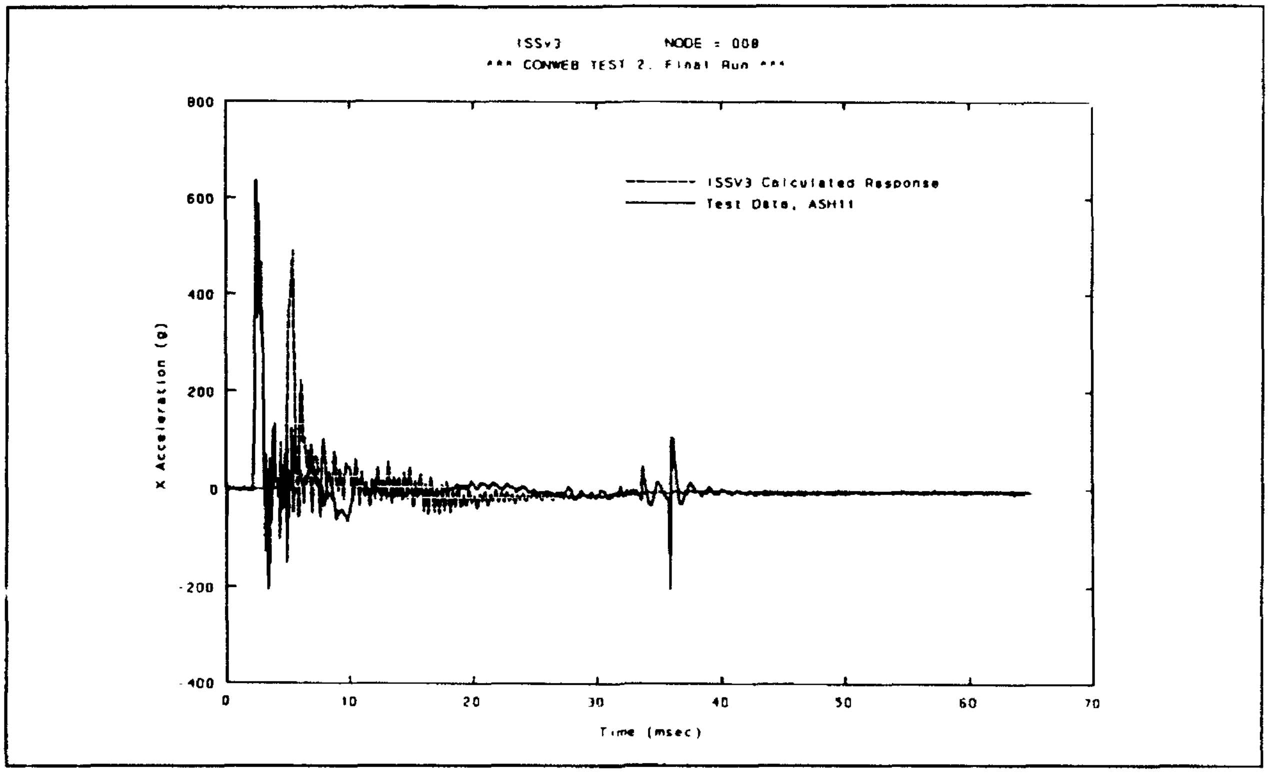

Figure 6. CONWEB 2, floor horizontal acceleration, ISSV3 Node 8 analysis versus test data

throughout the structure, given the difficulty of measuring rigid-body displacements with accelerometers.

CONWEB 2 vertical acceleration, velocity, and deflection histories were also reproduced reasonably well by the ISSV3 analysis. Extensive comparisons of the calculated and recorded motion histories were written by Dove (1991).

Shock spectra were generated from the velocity histories for each position in the structure (Figures 3, 4, 5, and 7). These spectra show a very good correlation between the calculated values and spectra generated for test data. The only consistent underprediction was for the rigid-body deflection as seen on the low-frequency portion of the spectra. This underprediction is thought to be due to a combination of the problem with the modeling of clay backfill in the analysis, and the problem of measuring late-time motions with accelerometers. The front wall response was slightly overpredicted, which implies that the model of the wall was softer than in the actual test. The modeling of the front wall as a one-way slab, neglecting all two-way effects, could account for this relatively minor effect.

The initial evaluation of ISSV3 as an in-structure shock analysis technique is favorable. Calculation of all four CONWEB tests (Dove 1991) with very dissimilar backfill materials gave reasonable results. The acceleration records generated had forms and magnitudes quite similar to test data.

It is acknowledged that the current free-field model in ISSV3 needs to be improved. Late-time velocities are sometimes underpredicted due to flow effects in clay backfill. Also, other research by Hayes (1989) has indicated that the characterization of backfill by seismic velocity, density, and attenuation coefficient may be overly simplistic. In spite of these problems in the free-field models, the interface loads generated by ISSV3 for the CONWEB tests were reasonable.

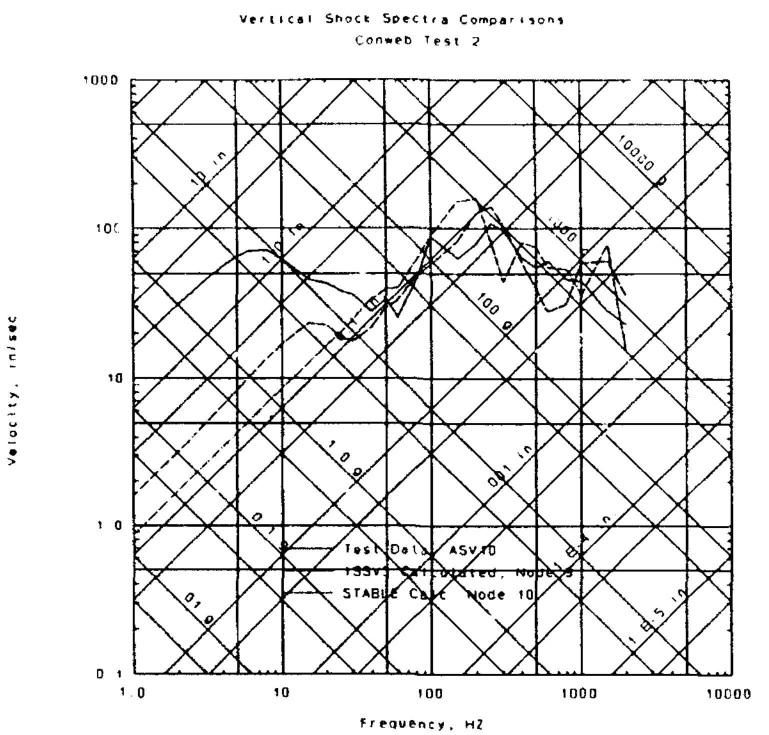

Figure 7. CONWEB 2, mid-floor vertical shock spectra, ISSV3 Node 9 comparison of analysis techniques

The single greatest strength of the ISSV3 program brought to light in the analysis of the CONWEB test series is the speed of the calculation. All of the calculations required less than 4 min to run on a 386, 20-MHz personal computer. This is a marked improvement over the 4 to 16 hr run times for this application of the STABLE program.

Comparison of Analysis Techniques

With the success of the initial evaluation of the ISSV3 program as an in-structure shock analysis tool, it is interesting to compare the program's results with the other available tools. Figures 3, 4, 5, and 7 show direct comparisons of the shock spectra generated from the CONWEB 2 series data and all of the means of analysis investigated. The average front wall response is compared to the TM 5-855-1 method, the SDOF wall analysis, and the ISSV3 results. The shock response on the floor of each structure is compared to the TM 5-855-1 method, the rigid-body motion SDOF analysis, the STABLE results, and the ISSV3 results. In every case, the results of the ISSV3 analysis are at least as accurate as the other techniques. With planned improvements in the free-field model, these comparisons are expected to be even better for later versions of the program.

It is important to note that the simplified in-structure shock methods investigated here are limited in a way that ISSV3 is not. The TM 5-855-1 method and the rigid-body SDOF analysis are best suited to simple, small structures. There is no fundamental reason why ISSV3 can not be applied to the fast and accurate in-structure shock analysis of larger, complicated, multifloor, multibay structures.

Conclusions

ISSV3 has been shown to be the best in-structure shock analysis tool examined. Extensive comparisons of the results of the in-structure shock analysis of the CONWEB test series, using several available methods, show that ISSV3 is fast, accurate, and easy to use. The speed with which ISSV3 can analyze

a given problem will allow the user the flexibility to quickly complete numerous in-structure shock calculations. This should encourage a designer to include in-structure shock considerations in the early design phase of a protective facility and have a direct impact on total design quality.

Acknowledgments

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The analysis contained herein was conducted at the US Army Engineer Waterways Experiment Station (WES) by members of the staff of the Structural Mechanics Division (SMD), Structures Laboratory (SL). This work was accomplished during the period October 1989 through March 1991 under the general supervision of Messrs. Bryant Mather, Chief, SL; and James T. Ballard Assistant Chief, SL; and under the direct supervision of Dr. Jimmy P. Balsara, Chief, SMD. The Principal Investigator for this project and the author of this report was Mr. Richard C. Dove, SMD.

Acknowledgment is made to Drs. Jimmy P. Balsara, and C. Dean Norman, SMD, SL, who provided guidance and constructive criticism throughout this study, and Ms. Pamela G. Hayes, SMD, for the provision of the CONWEB test data. Acknowledgment is also made to Mr. Scott D. Campbell, JAYCOR, Vicksburg, MS, for his assistance with the STABLE program and Mr. Tom Slawson, ARA Inc., Southern Division, Vicksburg, for his assistance with the ISSV3 program.

COL Larry B. Fulton, EN, was the Commander and Director of WES during this study and the preparation of this paper. Dr. Robert W. Whalin was Technical Director.

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Effectiveness of Passive Airblast Attenuation Devices Against Conventional Weapons Effects

by
Randy L. Holmes[1]

Abstract

The Defense Nuclear Agency (DNA) has initiated a research program to document the feasibility of using passive airblast attenuation devices to restrict the passage of airblast pressure into the intake and exhaust systems found in existing semihardened and protective facilities. Under the sponsorship of DNA, the North Atlantic Treaty Organization (NATO), and the Office, Chief of Engineers, the US Army Engineer Waterways Experiment Station (WES) conducted three full-scale tests in an effort to determine the feasibility of using passive airblast attenuation devices subjected to close-in detonation of conventional weapons.

Test results indicate that, for conventional weapon effects, passive airblast attenuation devices are as effective as the presently used active airblast valves in reducing the peak airblast pressure and associated impulse to acceptable levels.

This paper discusses the test results for the active airblast valves and the passive airblast attenuation devices.

Introduction

The intake and exhaust systems furnish circulated air for human consumption and for use by equipment. Active airblast valves utilize moving parts to shut off or to attenuate the passage of airblast. Such valves are associated with high initial and high maintenance costs. Present active airblast valves were designed to provide protection from long duration nuclear blast effects rather than for the effects associated with conventional weapons. As discussed in Headquarters, US Army Force Engineering and Services Center (1989 Jun), recent tests sponsored by DNA have shown that many of these active devices failed when subjected to the high-peak pressures associated with conventional weapons effects—consequently, jeopardizing the mission of the facility.

In order to demonstrate the effectiveness of passive devices, three full-scale tests in which both active valves and passive devices were subjected to airblast from a rocket and two general-purpose (GP) bombs were conducted at the DNA test site at the White Sands Missile Range, White Sands, New Mexico.

During each test, two passive devices and two active valves were used. A total of four different passive devices and two different active valves were evaluated. The passive devices included a one-plate and a six-plate orifice design developed by Wilfred Baker Engineering (WBE) under an SBIR Phase 1 contract with WES.

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The other two passive devices were provided by the LUWA Company. The LUWA Company, along with the TEMET Company, provided an active valve to be evaluated during these tests.

Discussion

The project task included design and fabrication of a reusable reaction structure and expansion chambers that modeled the air intake and exhaust systems of existing semihardened and protective facilities. The reaction structure was a 4-bay-wide by 3-bay-deep, 41-foot 4-inch by 35-foot 4-inch heavy steel-framed structure constructed on a reinforced concrete floor. The building was reinforced by shear and perimeter wall panels. The design of the structure is discussed in Holmes (in preparation).

Four expansion chambers were placed on top of the reaction structure. Each expansion chamber housed a passive airblast attenuation device or an active airblast valve. Design constraints for the passive devices are discussed in Baker (1989), and consisted mainly of the following: pressure attenuation below the maximum air intake level of 2 psi, valve size, cost, pressure and impulse attenuation of 100:1 or greater, and airflow characteristics (Table 1). The passive devices must provide protection for personnel and equipment while not adversely affecting the effectiveness of the air handlers and other ventilation system components; therefore, the pressure drop across the passive devices must be minimized. Detailed information about the passive devices and active valves are provided in Baker (1989), "LUWA Attenuator Type AA" (1946), "LUWA Attenuator Type AS" (1946), "LUWA Explosive Protection Value F," and "PV Blast Valves."

Table 1
Airflow Characteristics

Type	Pressure Drop at an Airflow Rate of 470 cfs
Passive devices	
WBE 1-plate	0.26 in. W.G.
WBE 6-plate	1.6 in. W.G.
Luwa type "AS"	2.2 in. W.G.
Luwa type "AA"	2.4 (I) in. W.G. 1.6 (E) in. W.G.
Active devices	
Luwa type "F"	1.1 in. W.G.
Temet type "PV-200-1"	0.6 in. W.G.

Note: W.G. indicates water gage and (I) and (E) indicate existing and entering of airflow, respectively.

The first test was conducted to determine the responses of the WBE passive airblast attenuation devices and the LUWA and TEMET active airblast valves to the detonation of a rocket. The rocket was detonated as a contact burst just outside the expansion chamber tunnel opening as shown in Figure 1. The airblast valves and devices used during this test are shown in Figures 2, 3, 4, and 5. The second test was conducted to determine the responses of the same devices and valves used during the first test to the detonation of a GP bomb. The bomb was located in front of the reaction structure as shown in Figure 1. The third test was conducted to determine the responses of the LUWA passive airblast attenuation devices and the same active airblast valves used in the first and second tests. A GP bomb was positioned at the same location as in the second test. The front wall panels sustained significant damage during the second test, requiring a sand-grid wall to be constructed in front of these panels to minimize the subsequent damage during the third test. The two airblast valves and the two devices used for the third test are shown in Figures 4, 5, 6, and 7, respectively.

A total of 28 channels of airblast pressure data were recorded during each test. Instrumentation layouts are shown in Figure 8. Gages were located on the side of the expansion chambers facing the explosive source and on the front and rear walls inside the expansion chambers. Gages were also located in the tunnel at corresponding locations as those located on the front walls inside the expansion chambers. The 12-inch-diameter galvanized pipe was instrumented at its entrance in the expansion chamber and at its

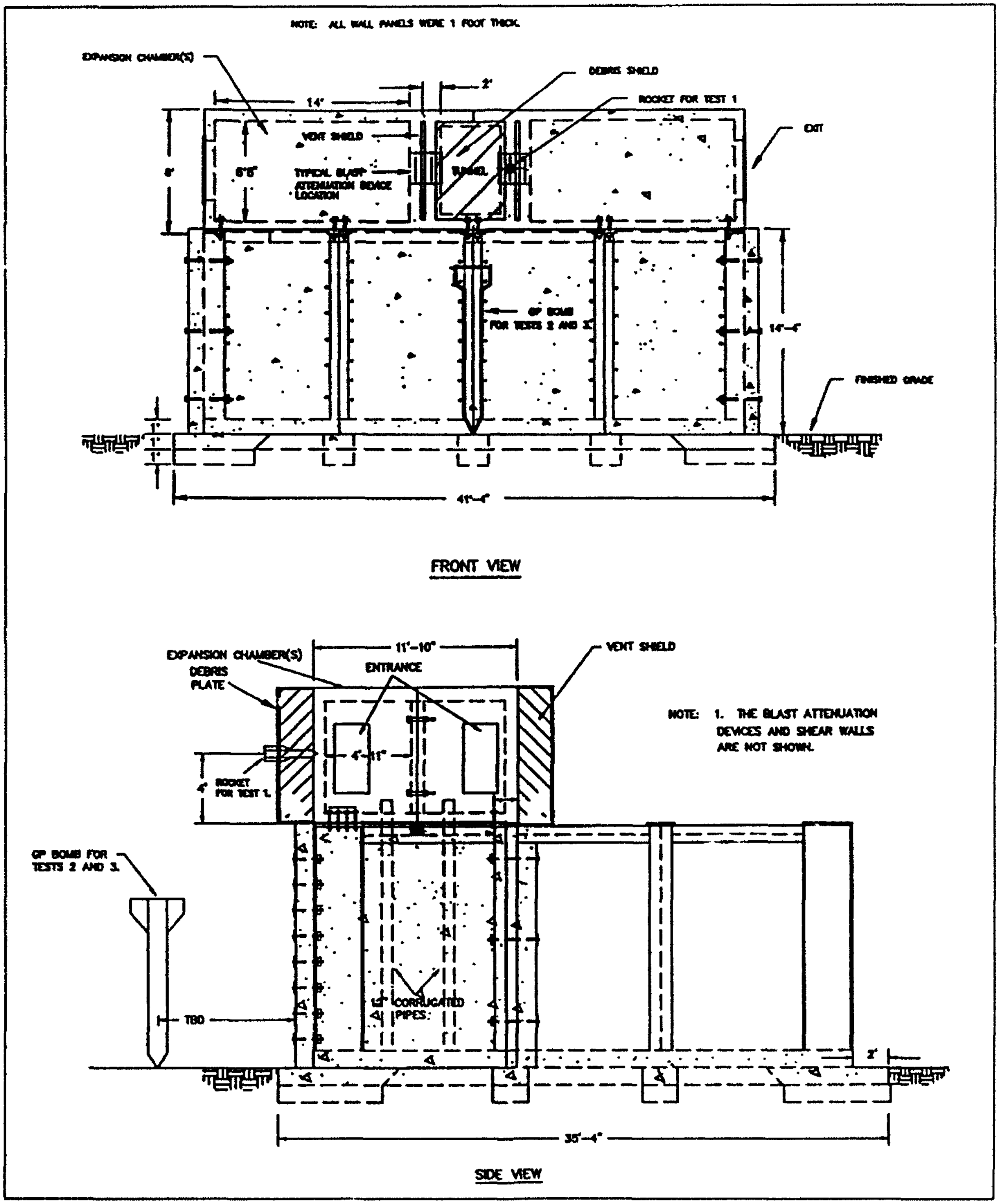

Figure 1. Test concept

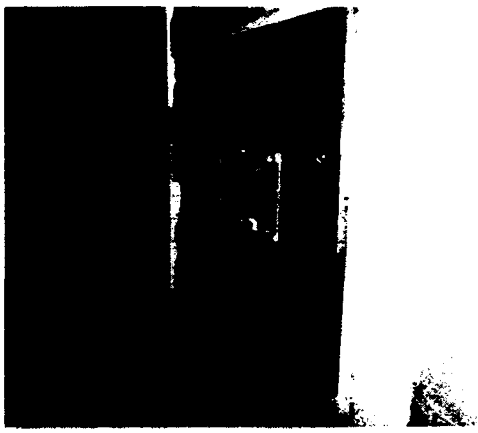

Figure 2. WBE 6-plate passive device, Tests 1 and 2

Figure 3. WBE 1-plate passive device, Tests 1 and 2

Figure 4. TEMET active valve model PV-60-300-1, Tests 1, 2, and 3

Figure 5. LUWA active valve type F, Tests 1, 2, and 3

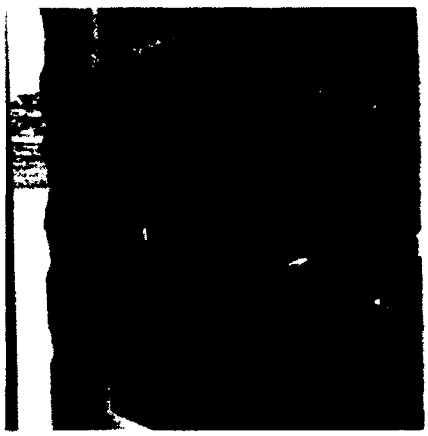

Figure 6. LUWA passive device type AA, Test 3

Figure 7. LUWA passive device type AS, Test 3

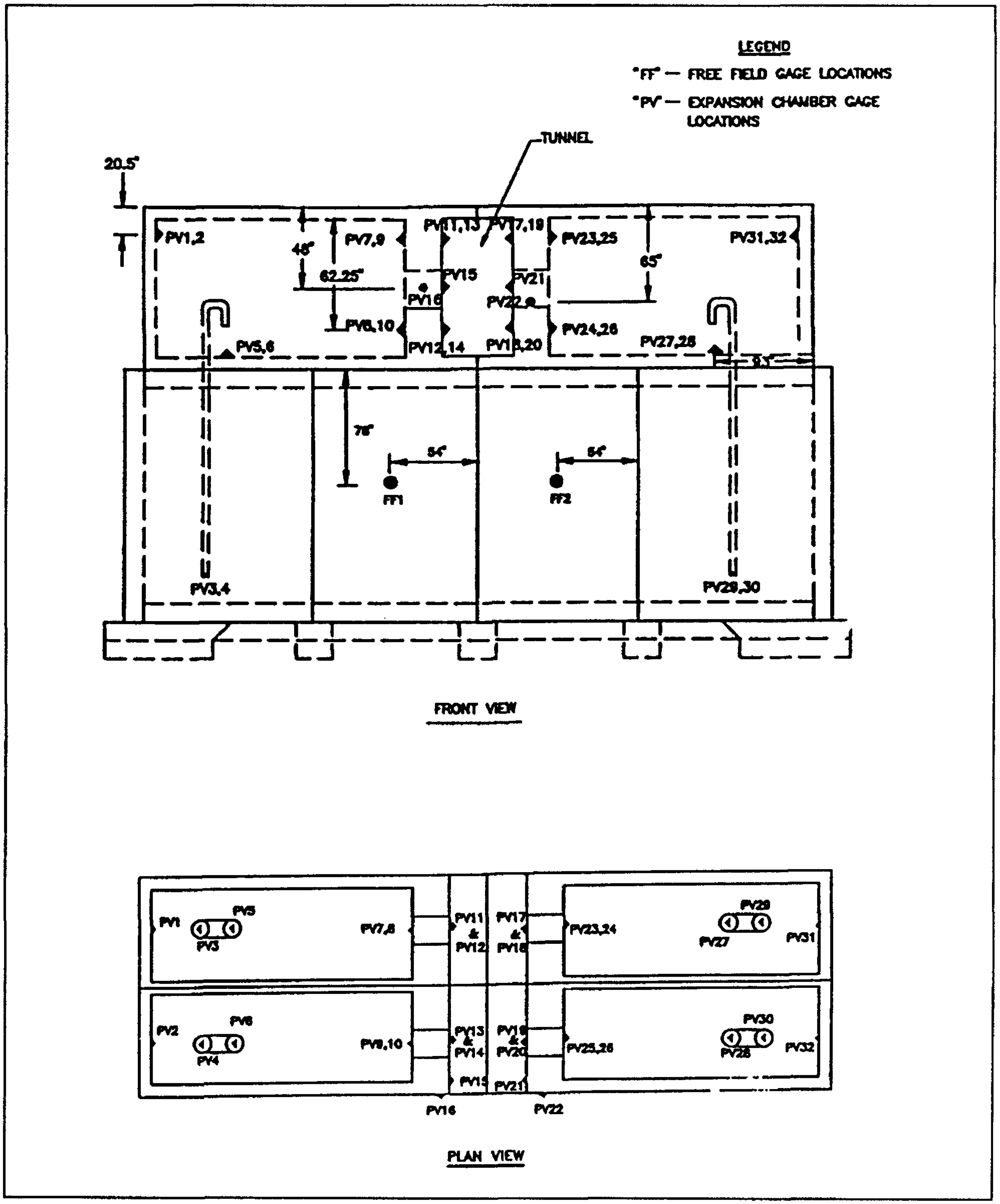

Figure 8. Instrumentation layout

exit in the room below. Two airblast gages were also located on the two center front wall panels of the reaction structure to measure the incident pressure. Airblast pressure is primarily attenuated by passing through the body of the passive airblast attenuation device, or the active airblast valve, and then expanding inside the expansion chamber.

During the first test, the highest pressure measured inside the expansion chambers occurred in the expansion chamber housing the WBE 1-plate passive device. The peak pressure was 5.7 psi and the associated impulse was 0.0161 psi-sec. The highest pressure measured inside the tunnel was 429 psi with an associated impulse of 0.191 psi-sec. This pressure was measured above the WBE 6-plate passive airblast attenuation device. The pressure measured just behind this device was 0.33 psi (the lowest pressure measured inside the expansion chambers), and the associated impulse was 0.0009 psi-sec. This device transmitted 0.08 percent of the peak pressure and 0.47 percent of the associated impulse into the expansion chamber.

During the second test, the highest pressure measured inside the expansion chambers occurred in the chamber housing the WBE 1-plate passive device. The peak pressure was 2.4 psi and the associated impulse was 0.004 psi-sec. The highest pressure measured inside the tunnel was 77 psi with an associated impulse of 0.0869 psi-sec. This pressure was measured in front of the WBE 6-plate attenuation device. The pressure measured just behind this attenuation device was 0.22 psi (the lowest pressure measured inside the expansion chambers), and the associated impulse was 0.0004 psi-sec. This device transmitted 0.29 percent of the peak pressure and 0.46 percent of the associated impulse into the expansion chamber.

The highest pressure measured inside the expansion chambers during the third test occurred in the chamber housing the LUWA-active valve. The peak pressure was 0.90 psi and the associated impulse was 0.0008 psi-sec. The highest pressure measured inside the tunnel was 75 psi with an associated impulse of 0.0799 psi-sec. This pressure was measured in front of the LUWA type "AS" attenuation device. The pressure measured just behind this device was 0.35 psi, (the lowest pressure measured in the expansion chambers). This device transmitted 0.47 percent of the peak pressure and 0.50 percent of the associated impulse into the expansion chamber.

A comparison of the passive and active devices results is shown in Table 2. This table shows the effectiveness of each device and valve in reducing peak pressure and associated impulse. Shown in Figures 9 through 14 are comparisons of airblast attenuation for the respective devices and valves.

Table 2
Comparison of Passive Airblast Attenuation Devices and Active Airblast Devices

Device	Peak Pressure		Impulse		Percent Transmitted	
	Enter	Exit	Enter	Exit	Pressure	Impulse
Test 1, Rocket Test						
6-Plate (P)	429	0.33	0.1910	0.0009	0.08	0.47
1-Plate (P)	155	5.70	0.2290	0.1610	2.63	1.32
Temet (A)	98	1.00	0.1060	0.0015	1.02	1.42
LUWA (A)	292	2.2	0.2200	0.0012	0.75	0.50
Test 2, First GP Bomb Test						
6-Plate (P)	77	0.22	0.0869	0.0004	0.29	0.46
1-Plate (P)	72	1.60	0.0453	0.0031	3.23	4.00
TEMET (A)	36	0.46	0-.7650	0.0009	1.37	1.13
LUWA (A)	71	0.86	0.0747	0.0005	1.20	0.67
Test 3, Second GP Bomb Test						
LUWA "AA" (P)	56	0.58	0.0644	0.0006	0.47	0.50
LUWA "AS" (P)	75	0.35	0.0799	0.0004	1.04	0.93
TEMET (A)	22	0.47	0.0663	0.0009	2.13	1.21
LUWA (A)	58	0.90	0.0776	0.0006	1.55	0.77

Note: "(A)" indicates active airblast valves.
"(P)" indicates passive airblast attenuation devices.

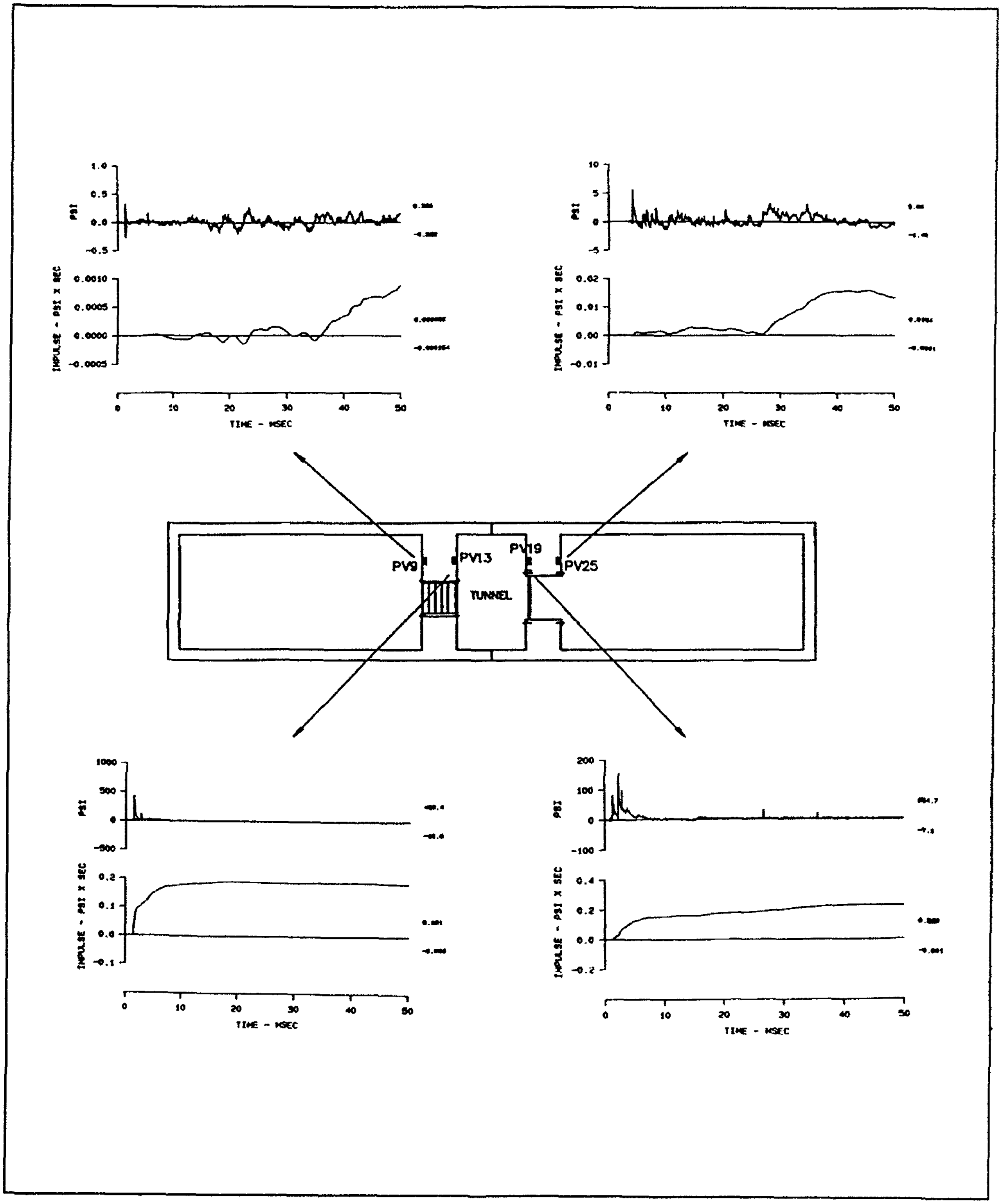

Figure 9. Comparison of airblast attenuation for the WBE 1- and 6-plate(s) passive attenuation devices for the first test

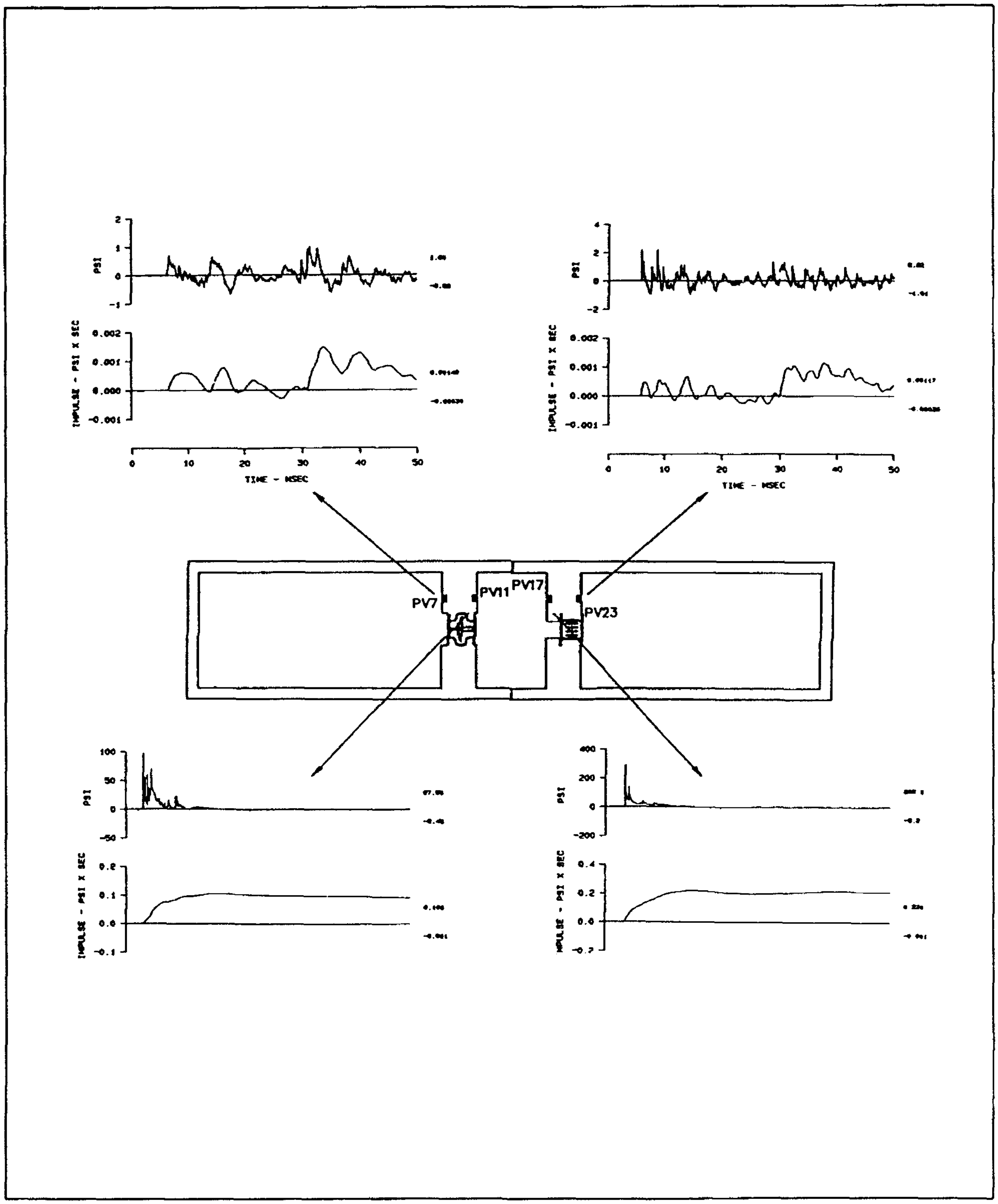

Figure 10. Comparison of airblast attenuation for the active airblast valves for the first test

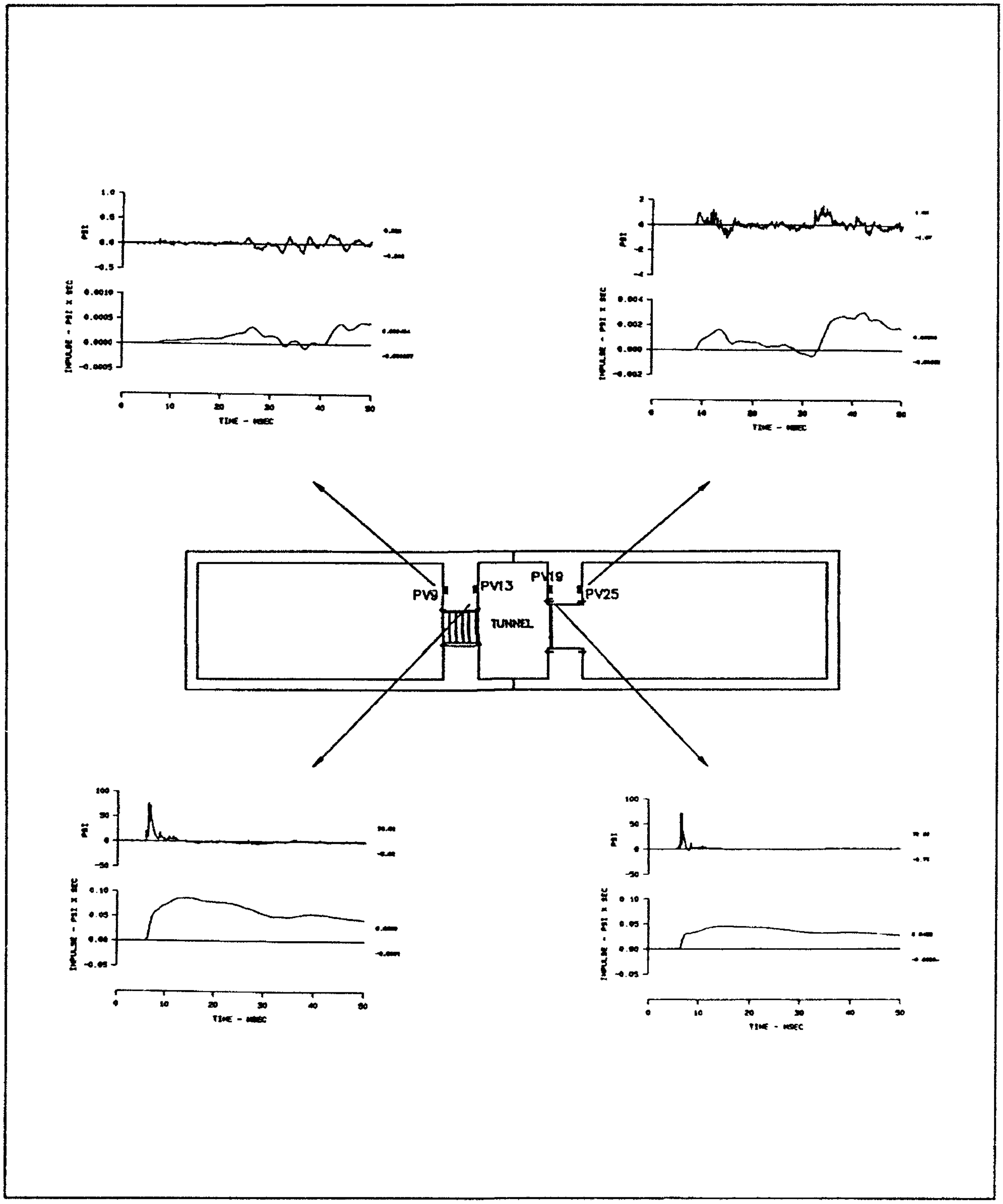

Figure 11. Comparison of airblast attenuation for the WBE 1- and 6-plate(s) passive attenuation devices for the second test

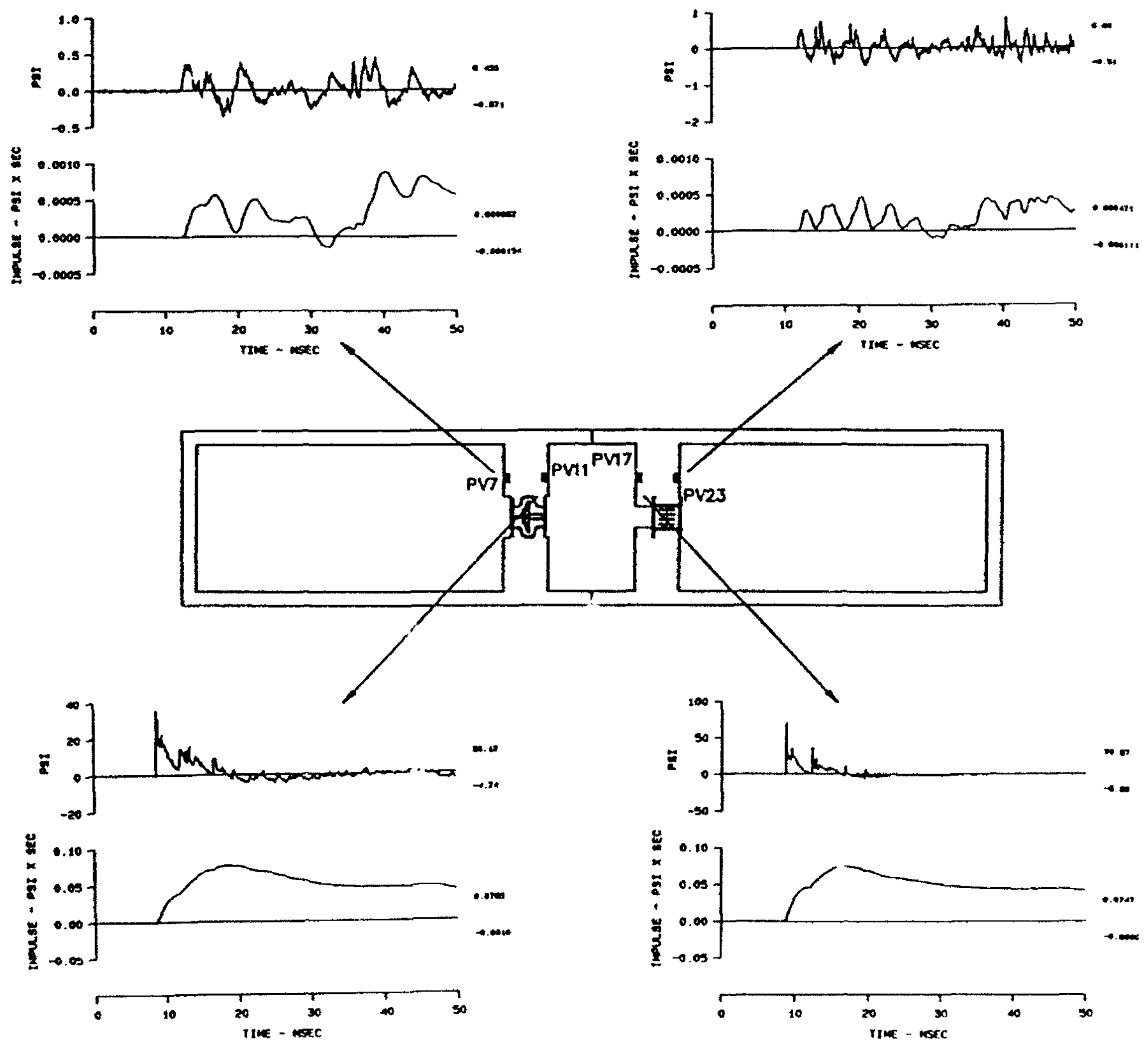

Figure 12. Comparison of airblast attenuation for the active airblast valves for the second test

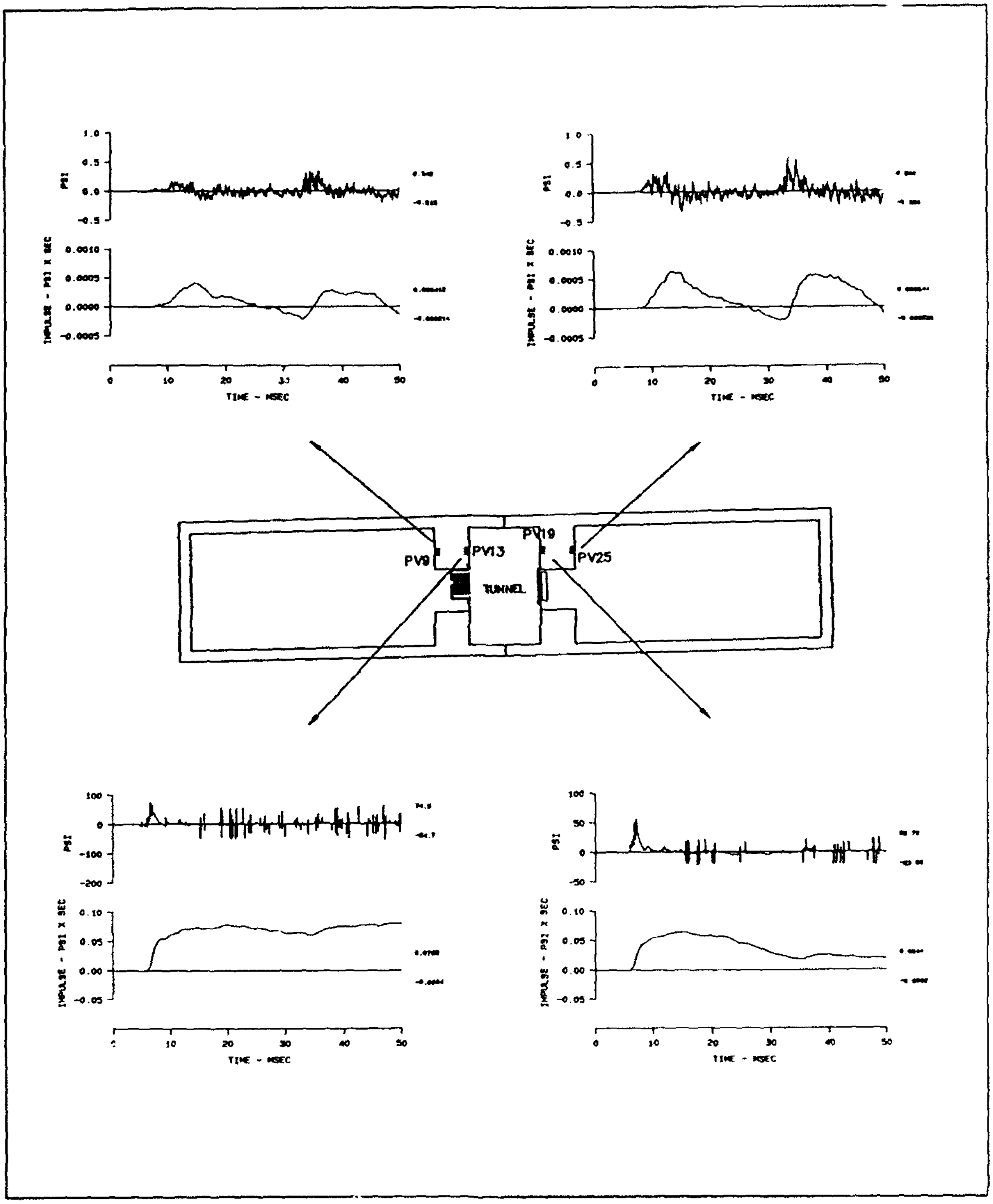

Figure 13. Comparison of airblast attenuation for the LUWA passive attenuation devices for the third test

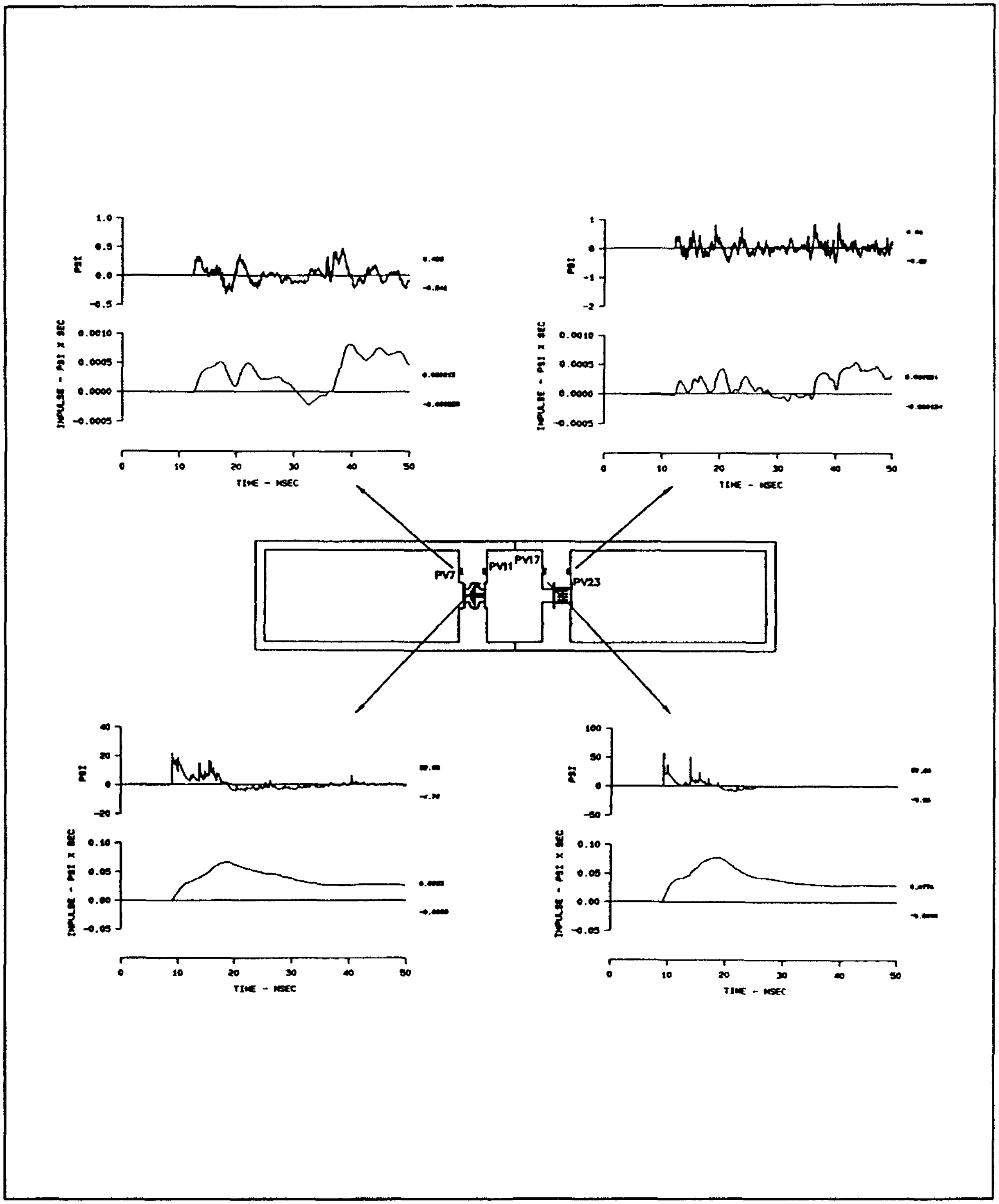

Figure 14. Comparison of airblast attenuation for the active airblast valves for the third test

Conclusions

Because of the large volumes associated with the air intake and exhaust systems in semi-hardened and protective structures, significant attenuation occurs before the airblast enters the expansion chambers through the blast valves. Further attenuation, if necessary, can be easily achieved by simple passive devices. For highly impulsive loads, passive devices offer a relatively low-cost and low-maintenance alternative to using active valves in semi-hardened and protective structures designed to survive conventional weapon effects.

The pressure and impulse transmitted through the passive devices and active valves were highly dependent on the peak pressure and duration of the input waveform. Test data indicated that all passive airblast attenuation devices and active airblast valves were effective in reducing the peak pressure in the expansion chambers. The WBE 6-plate passive device attenuated the peak pressure much more than required. A reduction in the number of plates would provide for the allowed airblast attenuation and at the same time proportionately improve its airflow characteristics. The two LUWA passive devices also attenuated the peak pressure much more than required. A redesign of the LUWA devices could provide the required airblast attenuation and improve their airflow characteristics. Because of the small frontal area of the LUWA valves, they can also be used in a stacked design that would enhance the ability of each device to attenuate pressure.

Test data indicate that a minimal amount of additional airblast attenuation was achieved after the airblast enters the expansion chamber following passage through the passive airblast attenuation devices and active airblast valves.

Recommendation

It is recommended that additional tests be conducted to determine the effectiveness of the expansion chambers.

Since little or no data exist for one to evaluate the effectiveness of the passive airblast attenuation devices subjected to long duration waveforms, additional tests should be conducted to determine the response of the passive airblast attenuation devices subjected to the effects of fuel-air explosive munitions.

Finally, no procedures are currently available for the designer to determine pressure, impulse attenuation, and airflow requirements for these systems. It is recommended that a PC-based computer program be developed to assist the designer in determining the proper passive airblast device for basic design constraints.

Acknowledgements

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Architectural and Design Features of the St. Peter Street Floodgates New Orleans, Louisiana: A High Profile Project

by
Alan D. Schulz, PE[1]

Abstract

The St. Peter Street floodgates are located at St. Peter Street and the riverfront in New Orleans, LA, in the heart of the French Quarter. The gates are part of the 8.3-mile floodwall that runs along the Port of New Orleans.

The project included construction of a reinforced concrete floodgate with two steel roller gates and related items. An innovative soil-founded design was selected for the floodwall to avoid pile driving and ground vibration problems in this highly developed tourist area. Spread footings monolithic with the base slab were used, in lieu of piling usually required for the area, to carry the design loadings.

Architectural features included floodgate columns patterned similar to the adjacent Jackson (Jax) Brewery shopping center. The columns were both a functional structural column with embedded metals and gate latching mechanisms, as well as an extension of the architectural features of the Brewery. Landscaping was added to help the project blend into the area.

Staged construction was required to meet the project constraints. Safety and informational signage were also used. Extensive coordination was required with affected owners and public agencies.

The design, architectural, construction, and communication features of this project will be presented to assist future designers with projects in congested urban areas.

For additional information, contact Alan D. Schulz, CELMN-ED-DD, Comm: (504) 862-2652.

1 Structural Design Section, US Army Engineer District, New Orleans; New Orleans, LA.

Project Funding and Authority

The St. Peter Floodgates Project is funded under the Flood Control, Mississippi River and Tributaries (FCMR&T) Construction Project. The floodgates are located in downtown New Orleans on the riverfront at St. Peter Street. In the heart of the French Quarter across the street from Jackson Square (Figure 1), the site is located two blocks from the St. Louis Cathedral and the Cabildo museums. The gates are part of the 8.3-mile floodwall system that runs along the Port of New Orleans.

Project History

The floodgates are located at the upstream end of the Dumaine Street floodwall on the east bank of the river at mile 94.6 AHP as shown in Figure 1. The Dumaine Street floodwall was built in the 1950's and provides concrete floodwall protection through most of the reach. St. Peter Street is situated at the upstream end of the Dumaine Street Floodwall. The Jackson Brewery, located at St. Peter and Decatur Streets, was then a brewery, prior to its 1984 development as a shopping center.

When the shopping center plan was developed, it infringed on the alignment of the flood protection as shown in Figure 2. The floodwall was realigned by permit, and new steel sheetpiling were driven along the present floodwall alignment to maintain the level of flood protection that existed before the development.

The Brewery development placed arch design panels (Figure 3) above the floodwall to form a riverside facade on the Brewery. The panel architecture had to be approved by historic preservation agencies. Precast architectural panels were suspended from the structural framing of the Promenade deck, which was an elevated, second-floor level walkway extending along the riverside of the shopping center. The precast architectural panels over the floodwall were designed to be removable for inspection and repair purposes.

Design Purpose and Environment

The design purpose for the St. Peter Street floodgates was to upgrade existing flood protection with a floodgate system in lieu of floodfighting and to improve vehicular and pedestrian access to the riverfront. Architectural design and landscaping had to be compatible with the historic French Quarter site. Safe day and night public use of the site had to be assured. Local interests had to be provided with an easily operated flood protection facility, and construction impacts on private and public facilities had to be minimized.

The US Army Engineer District, New Orleans, placed this system of two floodgates adjacent to the approved aesthetic arch floodwall design. Vehicular traffic patterns and pedestrian and handicapped access were improved in conjunction with the City of New Orleans Streets Department work. Special lighting for accent and safety was provided. The architectural and landscape design replicates the designs used in the adjacent Jax Brewery and Woldenburg Park. Three landscaped spaces (Figure 3) with Louisiana native trees and shrubs now enhance a once-open, barren area of shell, railroad tracks, and ballast. The area now provides shade and a green link connection to the adjacent Moonwalk and Woldenburg Park along the Mississippi River. The bottom roller gate design permits local interests easy operation and maintenance.

Construction Cost and Estimated Design Savings

The total cost of the construction contract was $309,277. The cost savings on foundation design was estimated to be $45,000. The project included construction of a soil-founded, reinforced concrete floodgate, two steel roller gates, demolition of part of the existing floodwalls, streets, sidewalk, and handrail. Construction started in September 1989 and was completed in May 1990. Professional Construction Services (PCS) of New Orleans, was the prime contractor.

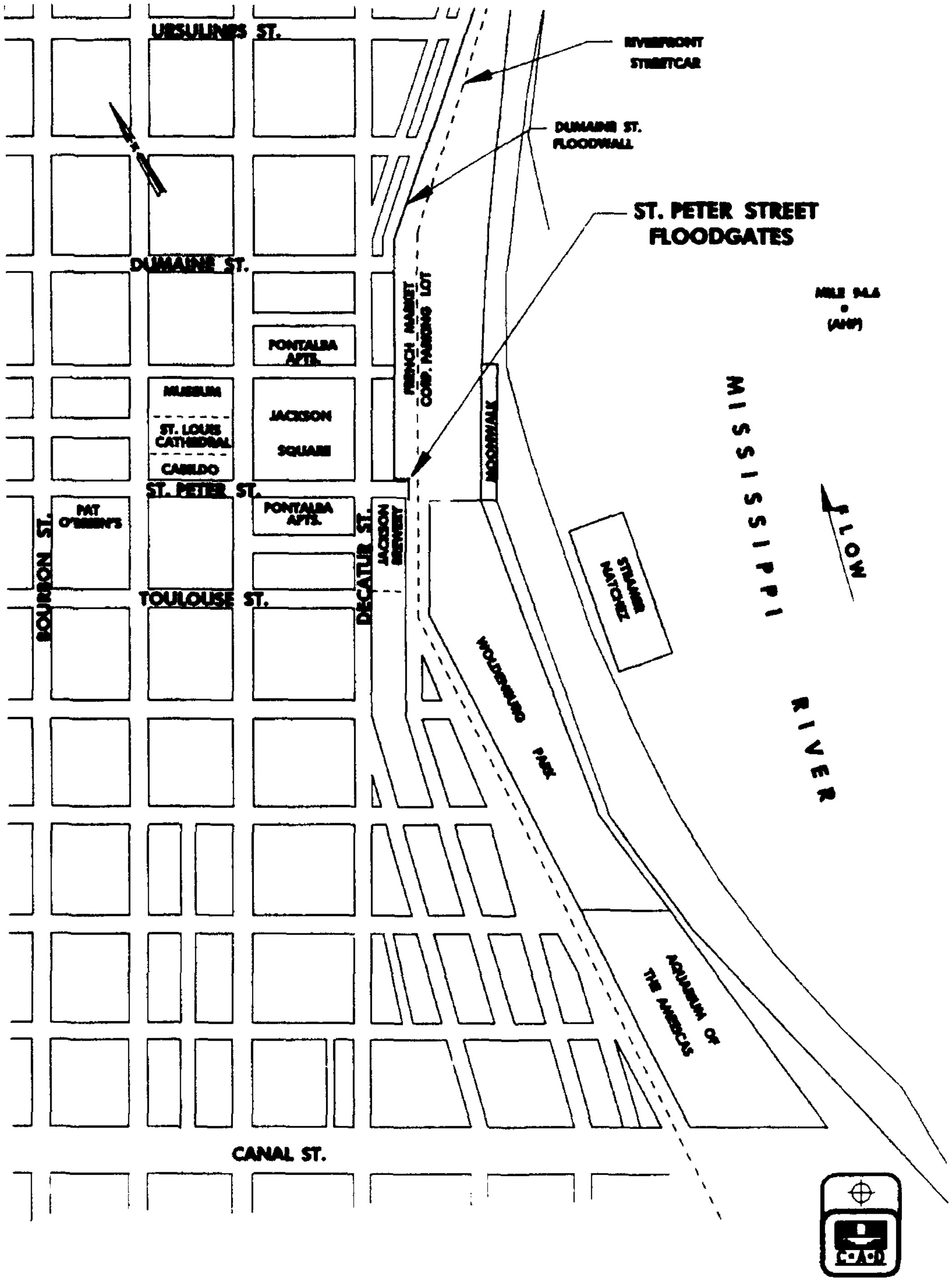

Figure 1. St. Peter Street Floodgates location map, downtown New Orleans

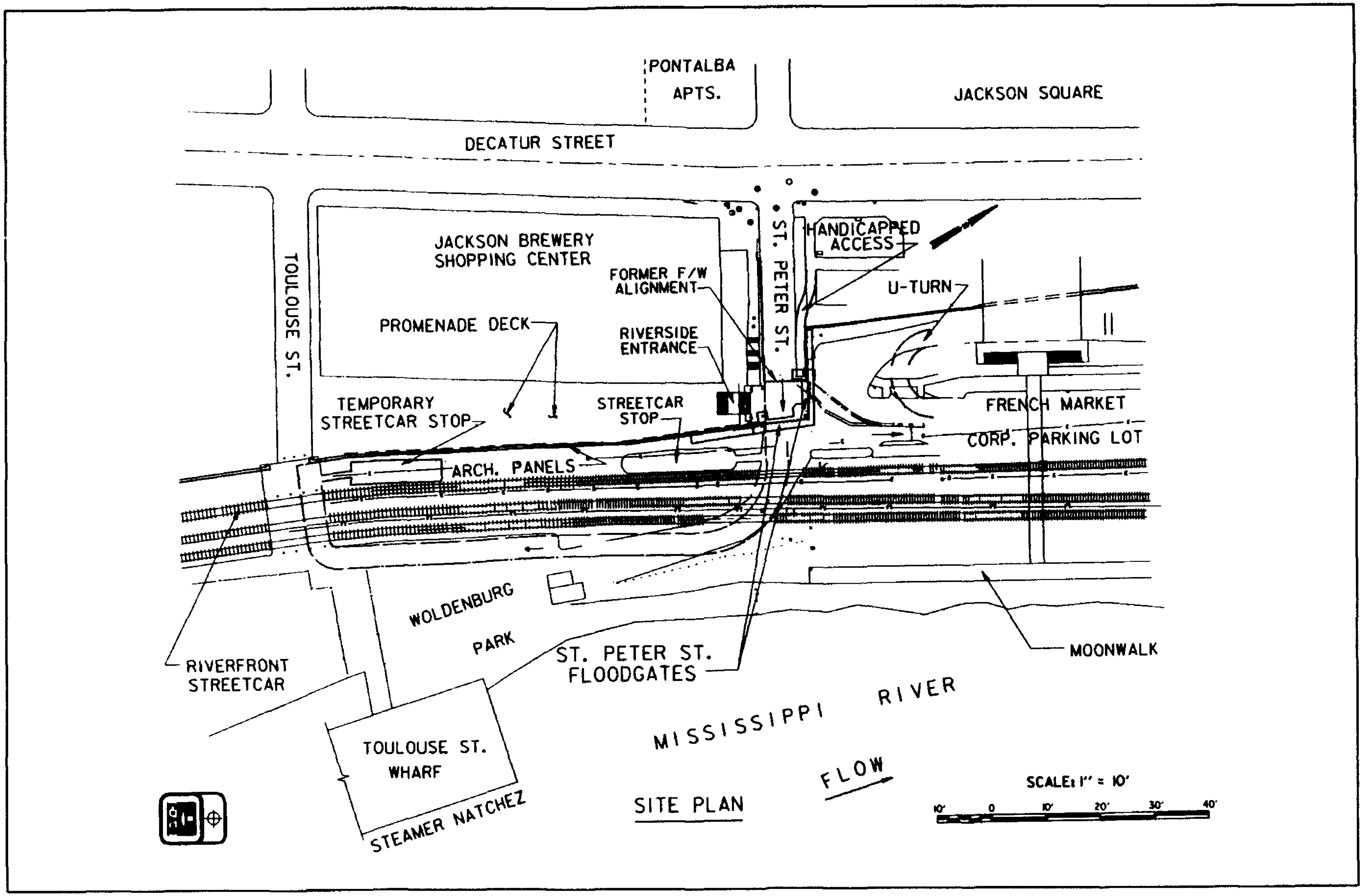

Figure 2. St. Peter Street Floodgates site plan, Toulouse Street to Moonwalk

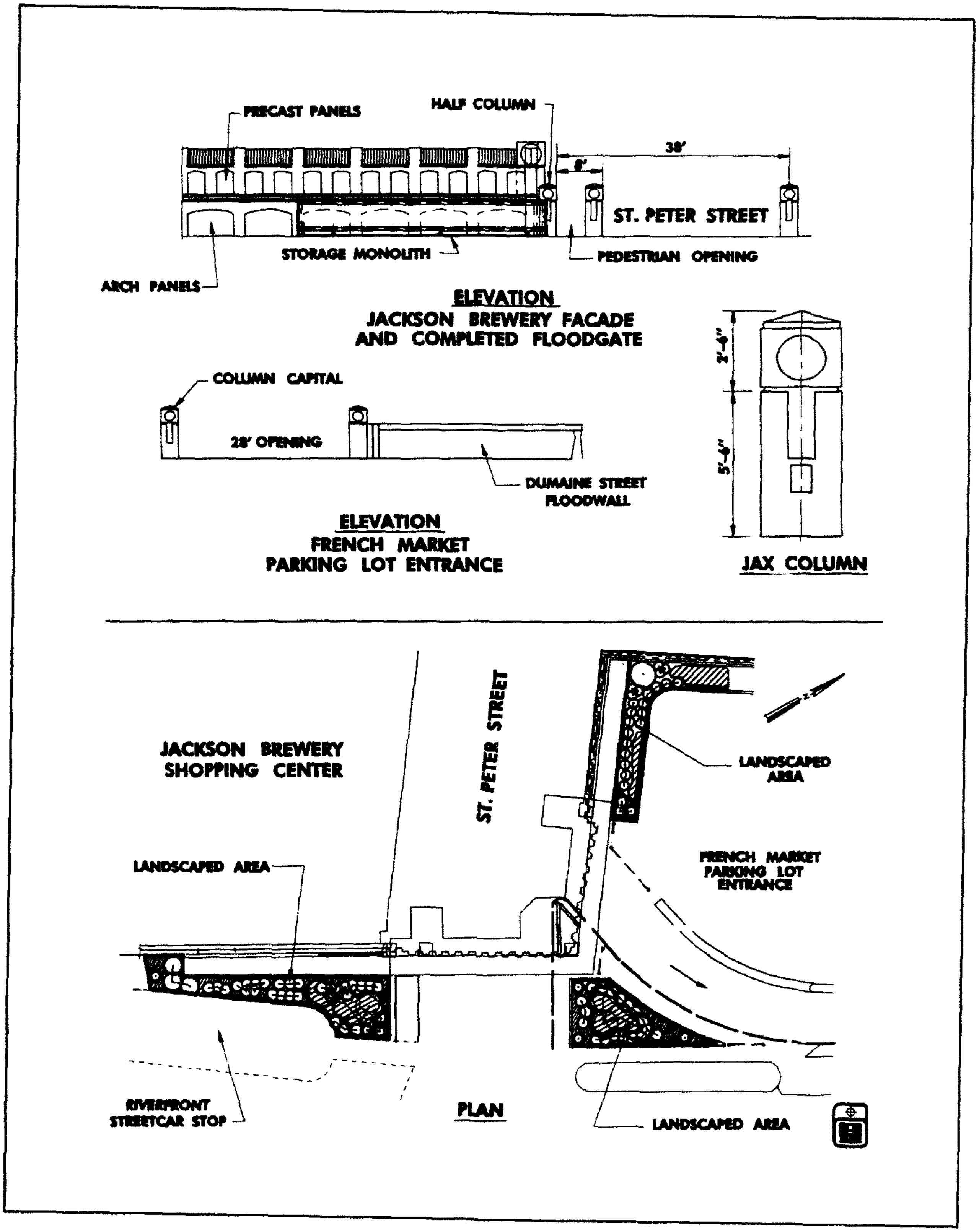

Figure 3. St. Peter Street Floodgates, architectural details

Architectural design features were coordinated by Drew R. Viosca, Supervisory Structural Engineer, New Orleans District (NOD). Landscaping was designed by Stephen F. Finnegan, Landscape Architect, NOD. The structural design was performed by Mark H. Gonski and Alan D. Schulz, Structural Engineers, NOD.

Innovative Design Features

Vibration risks/ foundation design

Instead of a pile-bearing foundation usually required for the area, a soil-supported spread footing foundation was selected. The design saved costs, reduced construction time, minimized disruption of local businesses and public facilities, and avoided possible pile driving damage to nearby structures. The historic Pontalba buildings are located one block from the construction site. The Jackson Brewery contains 75 individual shops, and most of the shops contain glassware and fragile items. A glass novelty shop located on the Promenade deck about 100 ft from the construction site is one such shop put at risk of damage by pile driving operations. The Promenade deck is not founded on a deep foundation, but on a shallow, mat-type footing that is susceptible to vibrations from pile driving.

Noise factor

Also, noise ordinances within the French Quarter limited the permissible hours of pile driving, due to the residential zoning of the nearby Pontalba buildings. Thus, pile driving for the floodgates was a high risk activity that had to be eliminated. Use of a monolithic, spread footing foundation in lieu of a pile foundation eliminated ground vibrations and substantially reduced construction risk and uncertainty.

Structural Design Features

Two favorable conditions existed for the floodgate structural design. First, the floodgate is L-shaped (Figure 4), and this permitted the designer to use the inherent strength of a corner in a monolithic base slab. The gate base slab was checked for design adequacy in two different ways. Each leg of the L-shaped gate was analyzed individually for overturning forces and resultant bearing pressures. Sliding and flotation forces were also analyzed. Then the L-shaped gate was checked for overturning about the skew axis of the L-shape. Centroidal axes and section properties of the L-shape were computed using the GAIP program (WES X0018).

Second, the floodwall height is only 5-1/2 ft high. The natural ground elevation at the floodgates is about 8 ft higher than average ground elevation in the area. Most floodgates in the New Orleans District are 8 to 12 ft high and require pile foundations. At St. Peter Street, the overturning forces and resultant bearing pressures were manageable with a soil-founded gate. A 2-ft-deep compacted shell base was provided underneath the floodgates and storage monoliths to provide a uniform, drained bearing surface.

The floodgates consist of two openings: 38 and 28 ft. The 38-ft opening spans 30 ft of St. Peter Street plus 8 ft of pedestrian walkway adjacent to the Brewery. To separate vehicular and pedestrian access, the floodgate opening on St. Peter Street was designed with an intermediate gate column. This also prevented the floodgate foundation from affecting the promenade foundation of the Jax Brewery. Inadequate space existed to allow a full gate column at the Brewery end of the gate. Thus, a half-column was constructed for the vertical gate seal at the Brewery side. The intermediate column was designed to take gate bearing. The half-column was architecturally treated to be visually the same size as the other columns. Each column had either a monolithic concrete capital or a removable composite capital to match the Jax column pattern (Figure 3).

The 38-ft-long steel gate was designed as a beam overhanging one support. Deflections at the end of the overhang were checked and found to be excessive. A stiffer, larger section

modulus gate beam was selected to limit end deflection. By using an appropriately sized rubber J-bulb seal, a positive seal was achieved.

The 28-ft span steel gate was designed as a simply-supported span bearing on the center column and on the Decatur Street (parking lot) side column. The center concrete column was designed to resist biaxial stresses imposed by the 38- and 28-ft gates. A soil-founded gate storage slab was provided for each gate.

An extension of the concrete base slab footing at each column permitted soil bearing pressures within the allowables. The dimensions of the extended base slab were determined by an iterative process until acceptable soil bearing pressures were achieved for all loading conditions.

Since the concrete base slab of the 38-ft gate opening had to be constructed in stages to accommodate continuous traffic flow, a vertical concrete joint was required at mid-length. Longitudinal reinforcing steel at the joint was spliced using mechanical couplers. Space limitations did not permit adequate length for tension lap splices. The vertical concrete joint surface was intentionally roughened to provide shear transfer and to achieve the maximum bond between surfaces. Sufficient reinforcing steel was also provided to handle torsional stresses.

Architectural designs were cast into the columns by using blockouts in the formwork. Embedded metal bearing plates, stainless steel seal plates, and gate latching mechanisms were placed in each column to blend into the architectural treatment.

A steel post and winch combination was designed to assist in closing the steel gates. The winch is mounted on a structural tube (TS) section which fits into a recess in the

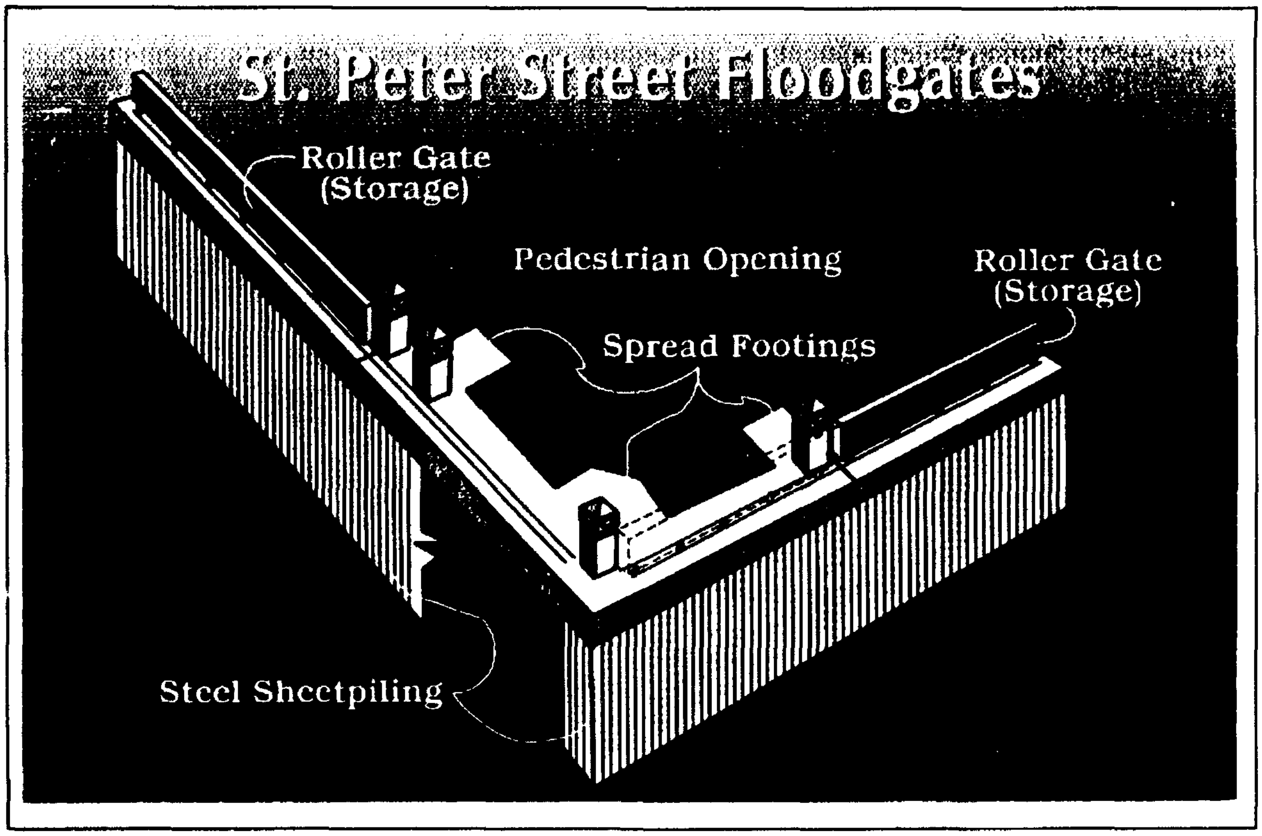

Figure 4. L-shaped floodgates

base slab near the center column. The recess is positioned to permit closure of each gate with the same winch.

Construction Features

A staged construction program allowed continuous vehicular access to riverfront tourist attractions and public parking. Public information displays, along with safety barriers, and fencing were used to tell the public about the work and keep them away from the construction at this heavily traveled area.

Three stages of construction were developed as shown in Figure 5. An enclosed area of safety fencing was used in each stage. The time duration of each stage was limited in accordance with agreements made with the affected parties. Roadway signage for vehicular and pedestrian safety was required in each stage.

Stage 1 consisted of setting up the field inspector's offic and preliminary work on the parking lot entrance. A U-turn area in the French Market parking lot was realigned to provide sufficient turning radius for vehicles in the new floodgate configuration. A time limit of 3 days was given for closure of the U-turn. Construction of a pilaster at the parking lot entrance for tie-in to the new floodwall was performed in this stage. The temporary streetcar stop at Toulouse Street was also started in this stage.

Stage 2 included construction of the gate at the French Market parking lot entrance monolithically with one-half of the gate across St. Peter Street. St. Peter Street was reduced to one lane by closure of the left lane with barricades and chain-link fencing. This stage blocked primary access to the French Market parking lot and was limited to 15 days. Alternate access, for passenger vehicles only, was provided during this 15-day period.

To meet the 15-day requirement, the floodgate concrete mix had to reach 3,000 psi in 3 days. This mix was specified in the design and the high, early strength was achieved easily in the field. A 3,000-psi mix at 28 days was used for the remainder of gate construction.

The storage monolith for the Decatur Street (parking lot) side gate was placed during this stage. Also, the temporary streetcar stop was completed prior to beginning Stage 3, which required closure of the existing station stop. The parking lot entrance and the left lane of St. Peter Street were reopened upon completion of this stage.

Stage 3 included closure of the right lane (Brewery side) of St. Peter Street and construction of the remaining one-half of the floodgate. The existing streetcar station was closed by fencing and the temporary station was opened. A plywood barrier was placed at the riverside entrance of the Jax Brewery. The concrete gate base slab tie-in was made using mechanical rebar splicers.

Both steel floodgates were installed during Stage 3. The Contractor was required to construct and store the gates offsite until the storage monoliths were completed.

Communication With Affected Owners and Agencies

Close coordination with 28 public and private interests during planning, design, and construction phases ensured an appreciation for the sensitive environment of the historic district. Coordination with the City of New Orleans Planning Commission and a host of other city departments was required. The major effects of floodgate construction were on the Jackson Brewery shopping center, the French Market Corporation parking lot, the Riverfront Transit Coalition Group Riverfront Streetcar, New Orleans Public Service (NOPSI) powerlines, the New Orleans Steamship Company, who operates the Steamer NATCHEZ, and the Aquarium of the Americas, which was under construction concurrently with the St. Peter Street Floodgates.

The floodwall was located next to the Jackson Brewery shopping center and construction affected the riverside entrance to the Brewery.

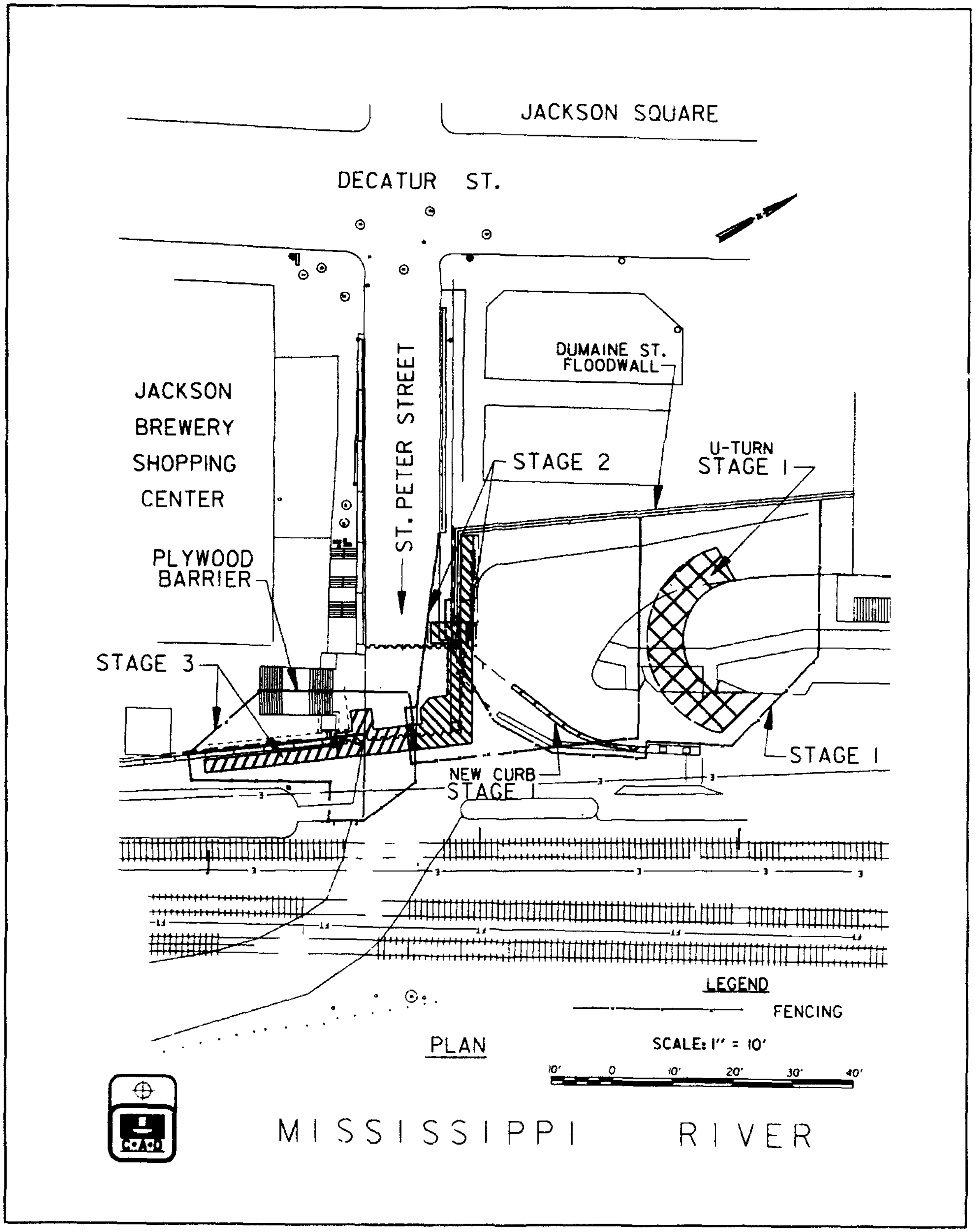

Figure 5. Three stages of floodgates construction

Bluestone pavers and French Quarter-style cast iron lampposts were removed and replaced to allow placement of concrete. The base slab of the floodwall extended beneath the Brewery sidewalk and came in close proximity to the Promenade foundation. Jax agreed to the temporary disruption of their riverside entrance and to electrical hookups of the lighting on the floodgate columns since the floodgate effectively extended the shopping center entrance. Jackson Brewery's project architects provided details of the Brewery specifications to ensure matching features in the floodgate elements.

Field construction was coordinated with the French Market Corporation and construction start-up was scheduled to avoid events such as the French Quarter Festival in June 1989. Also, the floodgate forms the entrance to the French Market Corporation parking lot. This lot is open all hours and could be closed during construction for only 5 min to install a new ticket dispenser.

The Riverfront Transit Coalition Group opened the Riverfront Streetcar line in August 1988 to coincide with the Republican National Convention opening in New Orleans. The Riverfront Streetcar runs past the construction site at 15- to 20-min intervals. The streetcar remained in operation during construction and thus could not be deenergized. A 600-volt overhead powerline for the streetcar was located 18 ft above ground and only 30 ft from the construction site. This was a safety concern and rotating construction equipment was restricted to avoid approaching the powerline.

The existing St. Peter Street Riverfront Streetcar station was temporarily closed, and an alternate station was built one block upriver at Toulouse Street. A concrete slab on grade was built along with a wooden handicapped access ramp to provide uninterrupted public transportation. The wooden ramp was treated with a skidproof coating for safety. Mercury vapor lighting was provided for the security of nighttime streetcar patrons. The Jax Brewery permitted the security lighting to be mounted on its riverside facade provided that the surface was restored after construction.

A NOPSI overhead, high-voltage power transmission line was located directly over the project site at a height of 60 ft above grade. This line could not be deenergized and was in the zone of pile driving leads if the floodgates had been pile supported. Elimination of pile driving avoided this problem.

The Steamer NATCHEZ is docked on the riverfront at the Toulouse Street wharf, which is one block upriver from the construction site. The primary access for NATCHEZ passengers is from St. Peter Street. The boat docked three times daily and tour busses and pedestrians entered via the St. Peter Street entrance. Also the sister vessel to the Steamer NATCHEZ, the riverboat BAYOU JEAN LAFITTE, left the dock daily for charters. Thus, continuous access through at least one lane of St. Peter Street was needed. A staged construction plan was developed to keep one 14-ft lane open during construction.

Construction for the Aquarium of the Americas and Woldenburg Park was concurrent with the design and construction of the St. Peter Street Floodgates. Coordination during design was required to ensure compatible access routes and to avoid interference of rights of way. Coordination between contractors was required in the specifications to facilitate field construction operations.

Handicapped, Pedestrian, and Vehicular Access

The configuration of the St. Peter Street floodgates was designed to provide sufficient turning radius for tour busses, delivery trucks, and fire-fighting equipment to go from St. Peter Street to the riverfront and public parking. Chain-lined bollards were used to keep vehicular and pedestrian traffic from interference. The Contractor was required to maintain an effective pedestrian and vehicular traffic management plan for each stage of construction. Temporary traffic rerouting was permitted where an

alternate route was available. A police officer was arranged to be on duty for traffic control during periods of construction when St. Peter Street was reduced to one lane. We advised the Contractor to take appropriate precautions in planning and executing construction operations to provide for the public safety.

Concurrent with the Corps floodgates project, the City of New Orleans Streets Department provided a handicapped access from Decatur Street through the floodgates and up to Woldenburg Park. The Streets Department designed a special off-street curved ramp, with an architectural handrail matching the Jax Brewery, to afford a safe and pleasing access to the riverfront green spaces. After completion of the St. Peter Street Floodgates project, the Streets Department installed a raised curb to permanently change St. Peter Street to one lane for safety and aesthetic purposes.

Architectural Features and Compatibility

Making the project "fit into the setting" was a prime concern for a floodgate system bordering the historic French Quarter. All elements in the design—the floodgates, color of paints, and the landscape plantings—were chosen to be compatible with other developments in the area. Concrete masonry columns were patterned after the promenade stairway columns of the Jax Brewery complex. Color and finish of the concrete surfaces match adjacent structures so closely, the viewer is unaware of the flood protection. The architectural purpose was to create a positive visual environment using structural elements and landscaping compatible with the area.

Planting beds are lined with old New Orleans street granite curbing. Architectural tree uplights and foot-lighted gate columns provide night lighting for public safety and increase aesthetic appeal for people going through the site to other riverfront developments. Underground electrical lines below the planting beds were placed in steel conduits encased with concrete treated with red dye to alert anyone digging in the area of danger.

The deteriorated pipe handrail on the St. Peter Street vehicular ramp leading to the riverfront was replaced with an architectural handrail that matches Jax Brewery. Nighttime lighting in the floodwall columns is the style of Jax Brewery. Landscape plantings consist of Louisiana iris, cypress trees, sage palms, nandina, monkey grass, Indian hawthorne, and elaegnus along with a cypress mulch. A buried irrigation system provides water to supplement rainfall runoff.

In summary, the total design sought to address all aspects of the French Quarter environment. In March 1991, the St. Peter Street Floodgates project received an Honor Award in The Chief of Engineers Design and Environmental Awards competition.

Conclusion

To achieve total design quality, the St. Peter Street floodgates included consideration of foundation and structural designs, safety measures, construction staging, communication with affected owners and agencies, handicapped, pedestrian, and vehicular access, and architectural features and compatibility within the confines of the historic French Quarter. The New Orleans District has sought to transform an ordinary floodgate design into an environmentally sensitive project while at the same time completing one of the last remaining gaps in the over 8-mile-long Port of New Orleans floodwall. Next time you are in New Orleans, please try to see it. Let us know your're coming and we'll have coffee and donuts waiting.

Precast Seal Beam for Railroad Closures

by
James Gunnels, PE[1]

Abstract

A combination levee/floodwall was built to protect the Wallsend area of Pineville, KY. This required crossing the C.S.X. railroad tracks at two locations. Because of the rapid rise of the river, gated structures were selected. For the gates to seal properly, a concrete sill beam was needed beneath the tracks. A conventional concrete structure would have a concrete base beneath the tracks with concrete piers on each side projecting out of the base and a gate hinged to one of the piers. Construction of the sill beam by cast-in-place methods would have taken the railroad lines out of service for an unacceptable period. Site conditions were not favorable to construct a run-around.

With these restrictions in mind, a sill beam of precast concrete that had minimal impact on railroad operations was designed. This design consisted of piers on each side of the tracks with their own separate bases and a precast concrete beam (beneath the tracks) resting on seats in the pier bases. H-piles were driven under each rail to help support the precast beam. These piles were driven with the railroad in operation (between the running of trains).

This innovative design allowed the construction of the closure structures without disrupting the railroad, saved the cost of a very expensive railroad shutdown, and saved the contractor time and the government money by not requiring extensive work to construct a run-around.

In April 1977, the Corps-designed floodwall at Pineville, KY, overtopped, and the city of 3,000 suffered $29,000,000 in damages. Fortunately, no lives were lost. Besides helping with disaster relief and recovery, the Nashville District began planning ways to upgrade Pineville's protection, which had been designed against a 20-year flood such as devastated the city in 1946. Incidentally, during the years those protective works were in place, their original cost of $2,000,000 was returned nearly fifteenfold in damages prevented.

The new plan called for raising the city's protection to SPF level. In conjunction with realigning and improving US 25 through the city, a new floodwall was built atop the relocated highway embankment, two bridges were replaced with higher structures, the interior flood control system was upgraded, the Wallsend levee was raised with a floodwall added along the crest, and nine closures were constructed. Outside the city, homes and businesses were evacuated, raised-in-place, or flood proofed against a 100-year flood.

[1] Structural Engineer, US Army Engineer District, Nashville; Nashville, TN.

Among the nine closure structures were two across the railroad tracks through the Wallsend levee/floodwall. These structures required some special engineering to achieve total design quality.

Initially, it looked as if the downstream railroad closure site would need a gate (or stop logs) 12 ft high and 105 ft wide across six tracks, each at different levels. When the railroad agreed to revise their tracks so that we would have only two lines to cross (and these at the same elevation), some major design headaches were removed, but a very important one remained—how to construct the closures without interrupting rail service.

Under normal conditions, two or three trains per hour pass through Wallsend, usually loaded with coal. The most construction time we could expect without interruption looked to be about 4 hr. With some rescheduling by the railroad and working on holidays, we expanded this to 18 hr, but here the railroad drew the line. If we were to avoid building a run-around, some innovative engineering would be needed.

The idea of a run-around(s) was considered but did not seem to offer much promise. At the downstream site, the closure is next to a radio station, a road, four siding tracks, and several residences. The upstream site is even more congested with roads, homes, the river, and a road underpass very close. Between the two sites to the north, the terrain rises very, very steeply. To the south is the Wallsend community itself. Clearly, the cost of a run-around looked prohibitive.

Considering again how we might construct the railroad closures within the time limits, we recognized that "near-track" elements like gate piers, floodwall, pier bases, etc., could be built within approximately 8 ft of the nearest track center line irrespective of schedule since shutting down the track would be unnecessary. But what about the items that crossed beneath the track: cutoff piling, bearing piles, and most importantly, the sill beam? A solution emerged in the form of a precast sill beam.

By the time all the design details were worked out, we had a 3-ft-wide by 3-ft-deep steel-reinforced precast concrete beam supported on H-piles where each rail crossed. At each end, the beam sat in a recess in the pier base. To get this beam and all accessories in place in the time available, our construction sequence was as follows:

- Railroad crews relocated all overhead signal lines into ground-level conduits laid parallel to the tracks to remove overhead obstacles to cranes.

- Sheetpiling was then driven parallel to the tracks to retain the material beneath the tracks during excavation for the pier bases (Figure 1).

- Next, the pier bases, piers, floodwalls, sheet-pile cutoffs, H-pile supports, etc., were constructed outside the sheet-pile containment (Figures 1, 2, and 3).

- While this construction was going on, a railroad crew was fabricating 15-ft-long removable rail sections for each track and adjusting tie spacings to accommodate pile driving. Working in "time packets" of 2 to 3 hr between trains, the contractor, in close cooperation with the railroad, was able to drive a line of sheetpiling perpendicular to the tracks (to act as a cutoff beneath the sill beam) and drove an H-pile to refusal under each rail along the center line of the sill beam. When a train was scheduled in the area, work ceased, the removable track sections were replaced, and the train stayed on schedule. After the train had passed, the sections were removed, and work resumed.

- To install the precast beam itself, 2- to 3-hr time packets would not do; so the railroad adjusted some schedules, and, by working on a holiday, an "18-hr window of time" was opened, and the most critical part of the work began.

  * The removable sections of rail and all ties within the sections were removed.

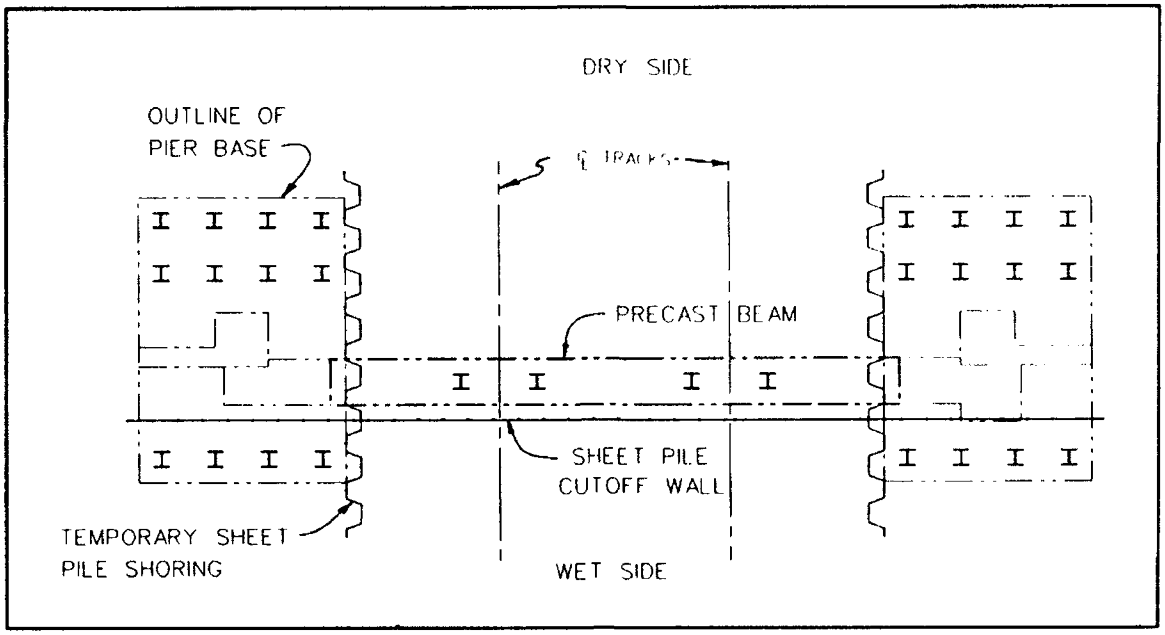

Figure 1. Foundation plan

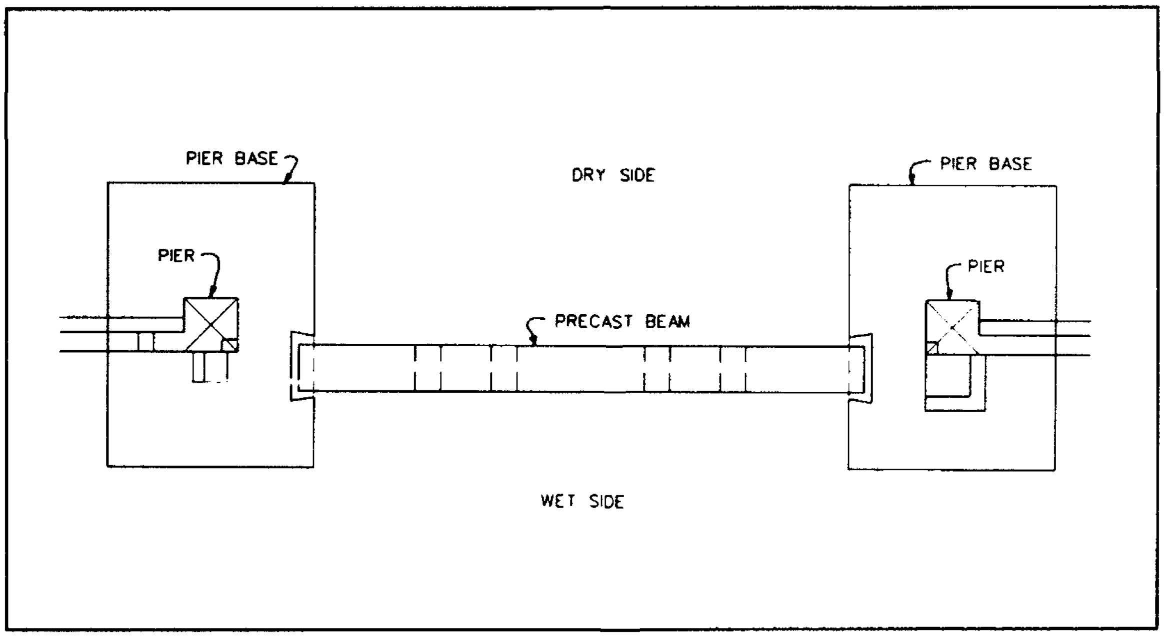

Figure 2. Plan view

* The area that was to receive the beam was excavated to a level sufficiently below the tops of the H-piles to allow the piles to be trimmed and fitted with load-bearing caps (Figure 4).

* The cutoff wall sheet piles had also been exposed by the excavation, and these were fitted with studs to tie them to the sill beam assembly (Figure 4).

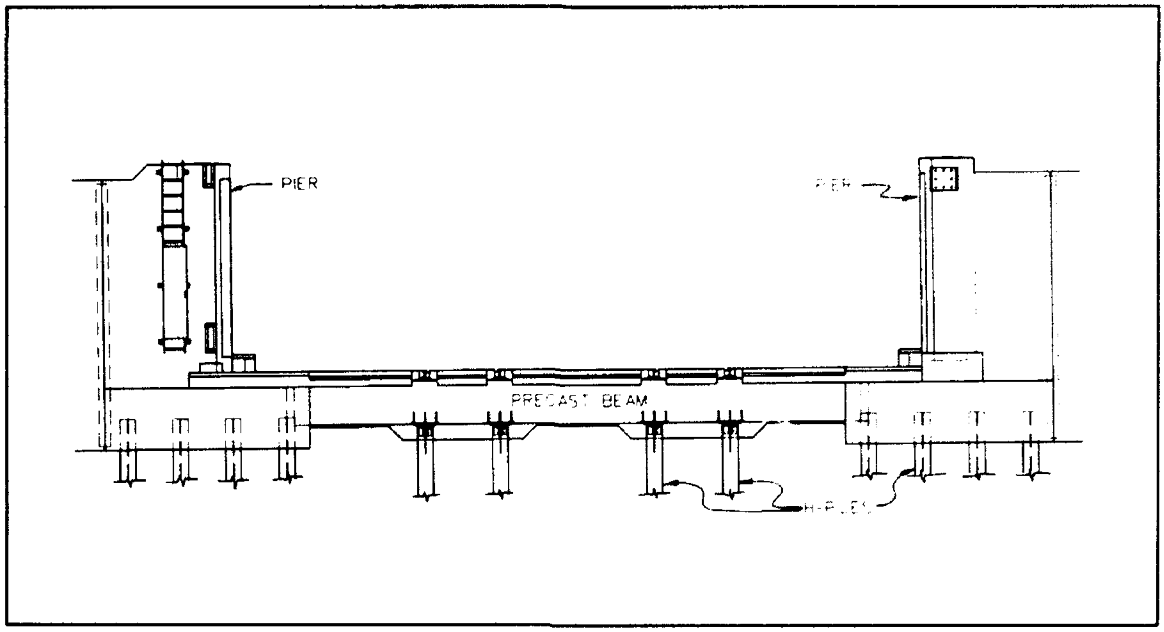

Figure 3. Elevation

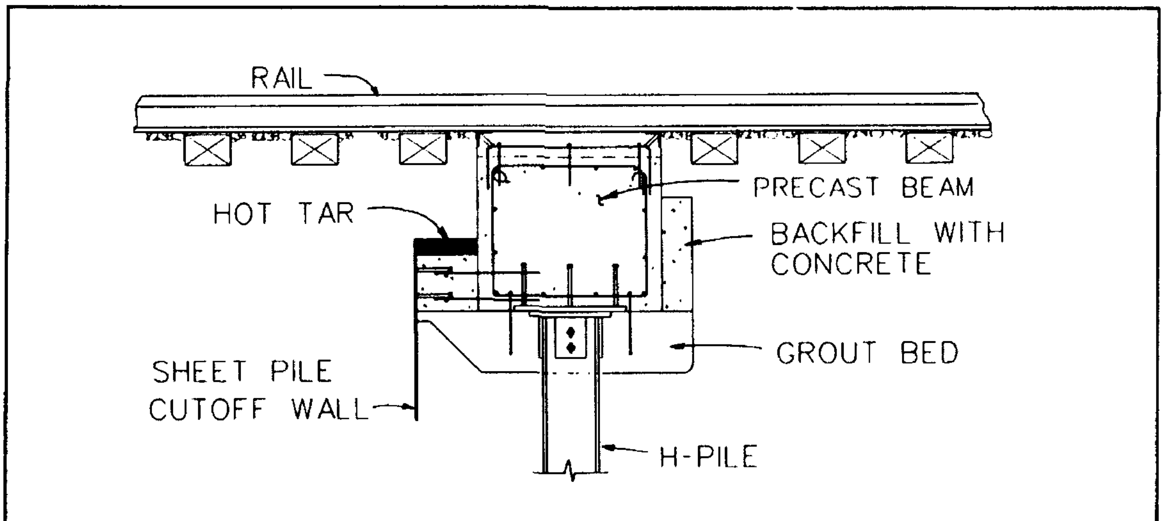

Figure 4. Cross section of excavated area

* Next the bottom of the excavation was filled with quick-setting mortar to the level of the pile cap/sill beam interface, and the sill beam was lowered onto the pile caps (Figure 4).

* This done, concrete was poured between the sill beam and the sheetpiling (engaging the earlier installed studs) and between the sill beam and the backside of the excavation. The concrete between the sill beam and sheetpiling was topped with hot tar (as a water barrier), and the rest of the excavation was filled to ground level with compacted ballast (Figure 4).

* When the rails and ties were replaced, the operation was ready once more to pass rail traffic.

- Now only the seal plate assembly and deflector plates remained to be installed.
  * First, threaded studs were welded to plates already embedded in the top of the precast sill beam (Figure 5).
  * Next, the seal plate itself (a T-section with clip angles) was positioned on the studs, and adjusting nuts were manipulated until the plate matched the bottom seal of the gate, which had been hung earlier so that this adjustment could be made (Figure 5).
  * This done satisfactorily, concrete was poured around the entire assembly (which included corner armor and anchors for the deflector plates), and extruded rubber blocks were fitted against the rails to form a water barrier (Figures 6 and 7). During closure, a wooden block was used to fill the small notch left for the train wheel flange.
  * The installation was completed when deflector plates (meant to protect the beam from impacts with items hanging from or being dragged by trains) were fastened to their already-embedded anchors. The installation went so well that when the contractor had finished, he had over 7 hr left before the next train was due.

Both railroad closures at Pineville were built with precast sill beams. Another one is under contract for the Harlan, KY, project, and two more (with stop logs instead of gates) are being specified for the protective works at Barbourville, KY. Using the precast sill beams for railroad closures shows how innovative thinking and sound engineering can produce **TOTAL DESIGN QUALITY**.

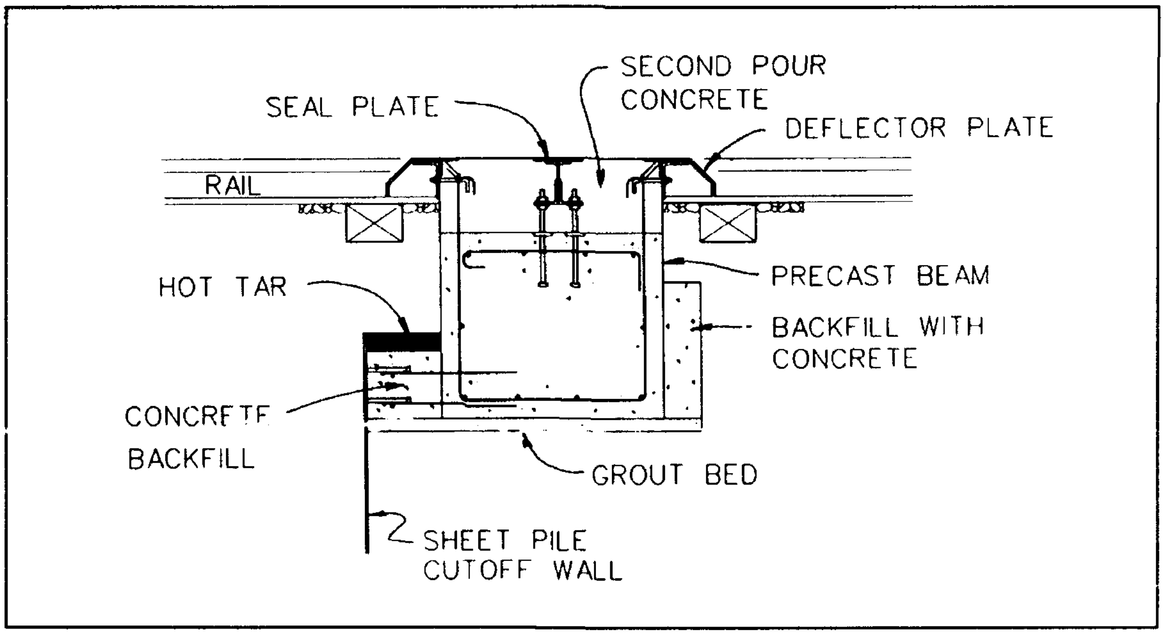

Figure 5. Cross section showing seal and deflector plates

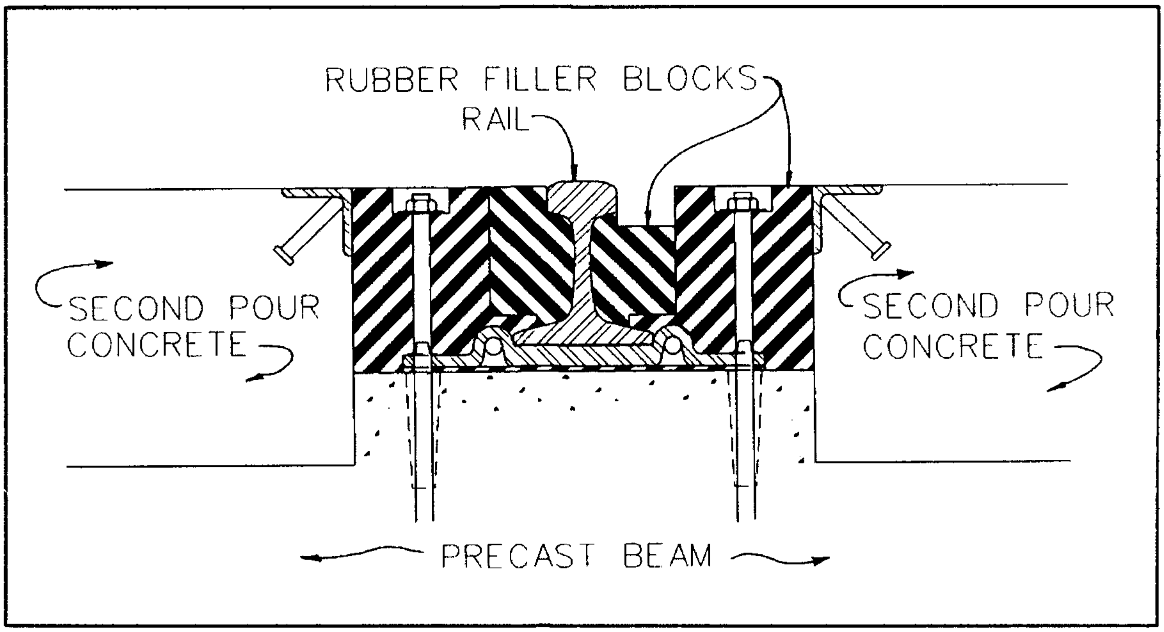

Figure 6. Concrete poured around assembly

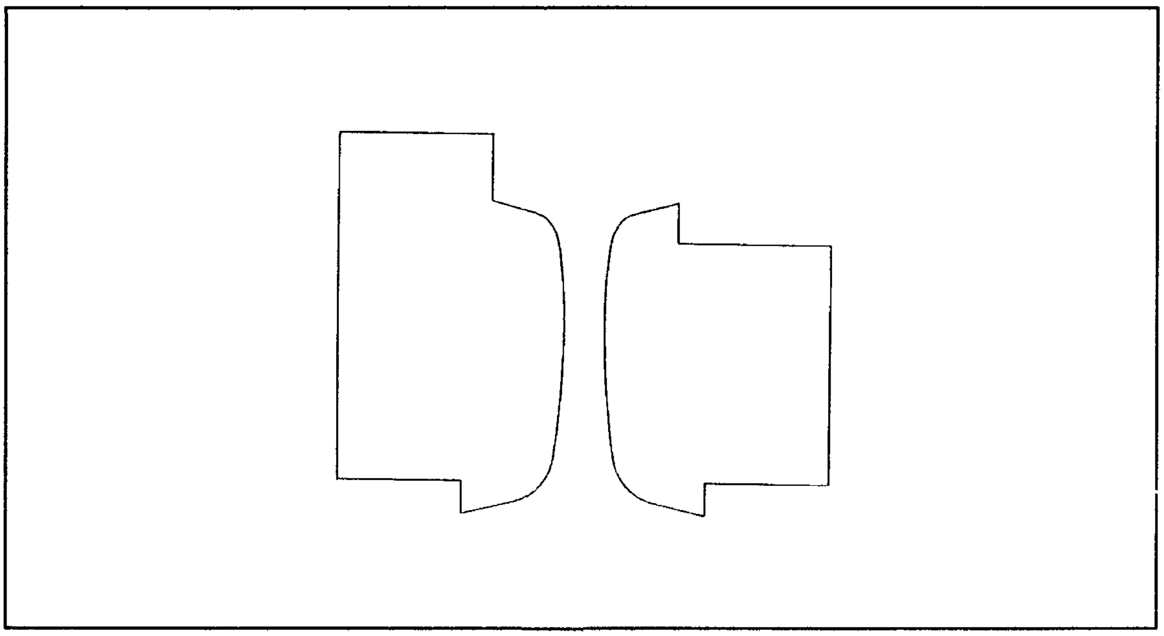

Figure 7. Rubber filler blocks

Tuesday—9 July 1991

Quality in the Constructed Project: ASCE Guide – Stephen C. Mitchell, Lester B. Knight & Associates

Structural Engineering Within Civil Works Direction and Priorities – Donald R. Dressler, HQUSACE

Recent Developments in Design and Analysis of Sheet Pile Walls – Reed Mosher, Information Technology Laboratory, US Army Engineer Waterways Experiment Station

Unique Soil Tie-Back Wall for the Oats Creek Flood Control Channel – John W. Hager, US Army Engineer District, Savannah

Design, Construction, and Evaluation of Earth Anchors – John Grundstrom, Chief, Geotechnical Engineering Section, and Ronald Erickson, Geologist, Geotechnical Engineering Section, US Army Engineer District, Detroit

Temporary Tieback Wall, Bonneville Navigation Lock – Dale F. Munger, Geotechnical Engineer, Patrick T. Jones, Geotechical Engineer, and Joseph Johnson, Structural Engineer, US Army Engineer Division, North Pacific

Mud Mountain Dam Intake Works Replacement – Paul C. Noyes, Structural Engineer, US Army Engineer District, Seattle

Analysis of the Seepage Cutoff Wall at Mud Mountain Dam – Wayne Kutch, Structures Design, US Army Engineer District, Seattle

Seven Oaks Dam Outlet Works Experiences Influencing Total Quality Design – Raymond Dewey, Structural Engineer, Structural and Architectural Design Section, US Army Engineer District, Portland

Tainter Gate Analysis – David J. Smith, Structural Engineer, US Army Engineer District, Omaha

Submersible Tainter Gate Addition to Peoria and LaGrange Wicket Dams – David R. Wehrley, Project Engineer, Design Branch, US Army Engineer District, Rock Island

Finite Element Analysis of Miter Gate Anchorage – Bruce C. Riley, US Army Engineer District, Pittsburgh

Structural Damages from Hurricane Hugo – Rick Lambert, Architectural/Structural Section, US Army Engineer District, Charleston

Structural Aspects of Caldwell Trucking Well No. 7 Superfund Site Design – William F. Strobach, Structures Section, US Army Engineer District, Kansas City

Ben Sawyer Bridge Restoration – Mark H. Nelson, US Army Engineer District, Charleston

Structures in the Blue River Channel, Kansas City, Missouri – Morris E. Ganaden, US Army Engineer District, Kansas City

Sheet-Pile and Precast Concrete U-Flume Low-Flow Channels for the Blue River Paved Reach Project – Kurt A. Mitscher, Structural Engineer, US Army Engineer District, Kansas City

London Avenue Canal Butterfly Valve Structure – Walter O. Baumy, Jr., US Army Engineer District, New Orleans

Miter Gate Diagonals, Stressing and Fatigue Concerns – Thomas J. Leicht, Engineering Divison, HQUSACE

Miter Gate Barge Impact Testing, Locks and Dam 26, Mississippi River – Cameron Chasten, Information Technology Laboratory, US Army Engineer Waterways Experiment Station, and Thomas Ruf, Engineering Division, US Army Engineer District, St. Louis

Miter Gate Sill Repair Caisson – Ralph B. Snowberger, US Army Engineer District, Louisville

Rehabilitation of the Santa Ana Bridge, Sandoval County, New Mexico – Lucy Ortiz, Structural Engineer, Architectural/Structural Section, US Army Engineer District, Albuquerque

Nonlinear Incremental Structural Analysis and Fracture Mechanics—A Logical Link – Barry D. Fehl, Structural Section, US Army Engineer District, St. Louis, and Kevin Z. Truman, Washington University

Nonlinear, Incremental Structural Analysis of Olmsted Lock – Sharon Garner, Structures Laboratory, and Chris Merrill, Information Technology Laboratory, US Army Engineer Waterways Experiment Station

The Yazoo Basin Demonstration Erosion Control Project – Jonathan W. Trest, US Army Engineer District, Vicksburg

Quadruple 84-Inch Corrugated Metal Pipe Repair – Tamara L. Atchley, Design Branch, US Army Engineer District, Detroit

Quality in the Constructed Project: ASCE Guide

by
Stephen C. Mitchell[1]

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1 Lester B. Knight & Associates.

Structural Engineering Within Civil Works Direction and Priorities

by
Donald R. Dressler[1]

Direction and priorities briefing agenda:

- Introduction.
- Project Design.
- Project Rehabilitation.
- Bridge Safety.
- Conclusions.

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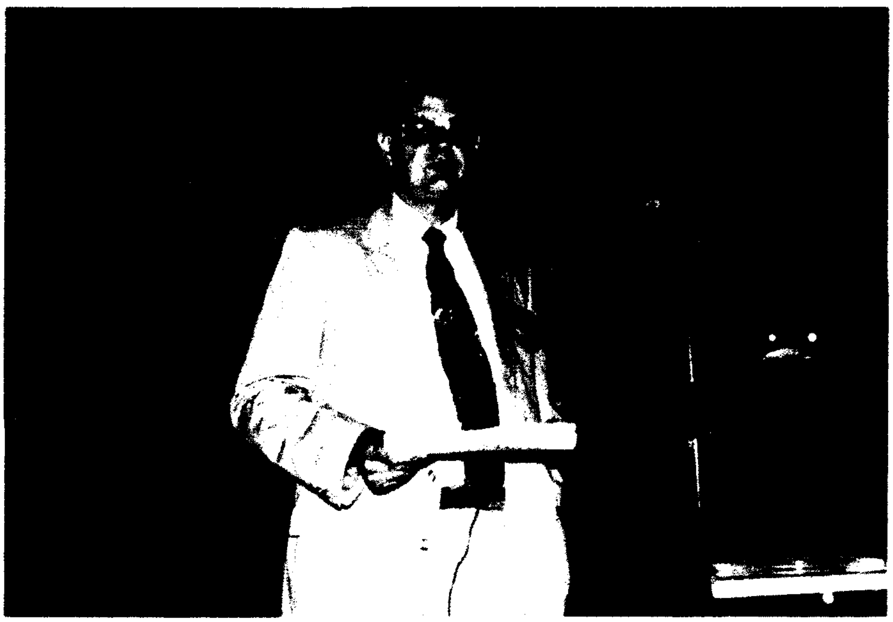
[1] Headquarters, US Army Corps of Engineers, Washington, DC.



Recent Developments in Design and Analysis of Sheet Pile Walls

by
Reed L. Mosher[1]

Abstract

Over the last four years a research effort has been under way at the US Army Engineer Waterways Experiment Station to investigate the fundamental behavior of sheet pile walls and develop improved design ans analysis procedures for sheet pile walls. The investigation initially studied the behavior of floodwalls. Miles and miles of sheet pile floodwalls have been constructed by the Corps. Generally the flood protection is provided by earth levees. Where the levee height is limited by weak natural soil conditions or levee width limitations, additional floodwall protection is furnished by sheet pile walls driven into the levee along its crest to form a floodwall. Traditionally, the required depth of sheet pile penetration and the sheet pile section have been determined using the classical design procedures for cantilever retaining walls. The classical design procedures are based on the assumption that the soil (after application of "factors of safety" or "strength reduction factors") is capable of exerting full "active" pressure or full "passive" pressure at every point except for a hypothetical "transition" zone near the bottom of the pile. The lack of reliable methods for calculating the limiting soil strengths encountered in these systems has led to what appears to be a very conservative selection of the final design penetration. The continuing reconstruction of existing floodwalls and anticipated new construction indicate the need for improved design techniques. This first step of the investigation involved the development and verification of a comprehensive analysis procedure for floodwall systems. The analysis process involved the formulation of a two-dimensional finite element model of the floodwall system including nonlinear soil behavior and allowing for separation of the pile and the adjacent soil. Verification of the process was made possible by the nearly unique luxury of the availability of results of tests of a prototype system. Once the analysis technique was verified with the prototype system, the technique was used to perform parametric studies for a variety of floodwall/levee configurations to obtain information on the general behavior of floodwalls and lay the ground work leading to the development of new soil-structure interaction analysis procedures.

The main purpose of the comprehensive finite element analyses was to understand the mechanisms involved in typical floodwalls in light of the results necessary to develop a simpler, yet reasonably accurate, soil-structure-interaction (SSI) technique suitable for design purposes. Initially, the possibility of deriving a set of

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"nonlinear soil response springs" was explored. Such curves, representing the relationship between the displacement and pressure at the wall-soil interface, could be incorporated in the conventional, one-dimensional SSI Winkler procedure. However, the finite element analysis revealed that the nature of the floodwall problem is more complex than can be handled in a one-dimensional model. Therefore, the second phase of investigation was devoted to the development of a new technique, tentatively called the "shear ring" method. In a third phase of the investigation, the "shear ring" method has been extended to the general case of sheet pile retaining walls, cantilever and single or multiple anchored walls.

This paper presents the finings from each phase and shows how this new procedure is superior to classical design and Winkler one-dimensional models. The impact of this research on the design of future sheet pile walls will be demonstrated.

Unique Soil Tie-Back Wall for the Oats Creek Flood Control Channel

by
John W. Hager, PE[1]

Abstract

The design of the Oats Creek Flood Control channel required enlarging the channel through the earth fill embankment of a six-lane divided highway. Several options were considered in the design, such as tunneling or cutting through the embankment and building a bridge over the channel. The Georgia DOT suggested a third solution. An existing bridge in the highway embankment for a railroad underpass offered an alternate channel route. This plan calls for excavating through the end fill of the bridge abutment under the first span of the bridge. The excavation requires retaining the end fill as the excavation proceeds. The norman design calls for driving shoulder piles for a tie-back wall; however, the bridge deck restricts head room and prevents driving. The original plan called for 8-ft, 6-in. shoulder piles placed in augured holes with 4-1/2-ft staged excavation zones. The total height of fill is 35 ft, requires six zones, with each zone tied back with soil anchors.

The Schnabel Foundation Company has proposed a unique method of constructing the soil tie-back wall. In the restricted head room area under the bridge, cast-in-place concrete shoulder piles will be used. The unusual feature of these shoulder piles is the hand dug wells which are 32 ft deep in which the concrete and reinforcement is placed. The design loading for this type construction is based on Terzaghi's theory of lateral soil pressure in lieu of Rankine's method. At the seminar the design and construction photos will be presented of the completed wall which will be under construction during the winter of 1990-91.

For additional information, contact John Hager, CESAS-EN-DS, telephone 912/944-5570.

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Design, Construction, and Evaluation of Earth Anchors

by
John Grundstrom, PE,[1] *and Ronald Erickson*[2]

Abstract

The US Army Engineer District, Detroit, has used earth anchors to tie back steel sheet-pile walls for more than 15 years. Recent anchor installations have had problems meeting the specified acceptance testing criteria. After a project at Holland, MI, the design, construction, and testing of earth anchors was reevaluated. This paper discusses the lessons learned and solutions developed during the Holland project.

Introduction

Earth anchors[3] are used to transmit force from a structural element on the surface into the soil. They are installed by drilling an inclined hole from the surface. The forces are transmitted to the surface by steel tendons (usually high-strength steel cables or deformed bars) which are inserted into the hole. The force is transmitted from the tendons to the soil by cement grout which is injected into the soil under pressure at the lower end of the hole. Figure 1 shows a diagram of a typical earth anchor installed for the Detroit District.

Earth anchors were developed and first used in Europe before World War II. In the 1950's, contractors in the US began using earth anchors as temporary construction tiebacks. However, it wasn't until the late 1960's and early 1970's that earth anchors began to be used for permanent construction in the US.

Detroit District's first project that utilized earth anchors was at South Haven, MI, and was constructed in 1971. Thus, we were among the first to utilize permanent earth anchors in the US. Since the South Haven project, we have used earth anchors at many other locations. The number of individual anchors installed on our projects is well over a thousand. Figure 2 shows the locations of Detroit District's anchor installations.

Detroit District Procedures

Typically, Detroit District uses earth anchors to anchor waterfront bulkheads. Often projects involve rehabilitation of 50+ year-old concrete and wood structures by driving steel sheet piles adjacent to them. See Figure 3 for an example of one our earth anchor installations.

Design

Our first earth anchors were designed because at South Haven Harbor there were some residences in close proximity to a water front bulkhead. In the early 1970's there was little

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[1] Chief, Geotechnical Engineering Section, US Army Engineer District, Detroit; Detroit, MI.
[2] Geologist, Geotechnical Engineering Section, US Army Engineer District, Detroit; Detroit, MI.
[3] While the phrase "earth anchors" is used by the Detroit District, ground anchors or soil anchors may be more common terminology. Earth anchors will be used in this paper.

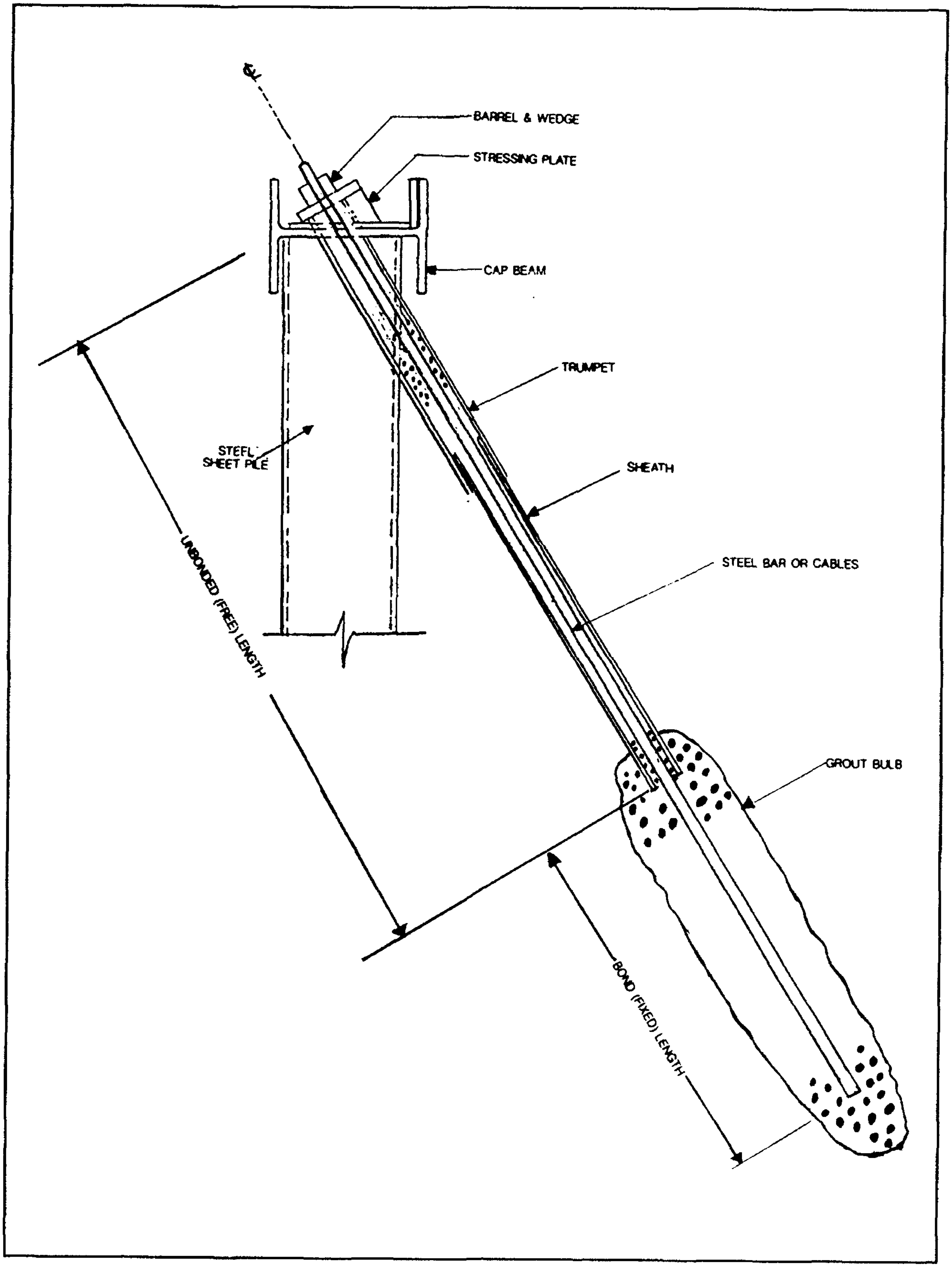

Figure 1. Typical earth anchor

Figure 2. US Army Engineer District, Detroit, earth anchors installation locations

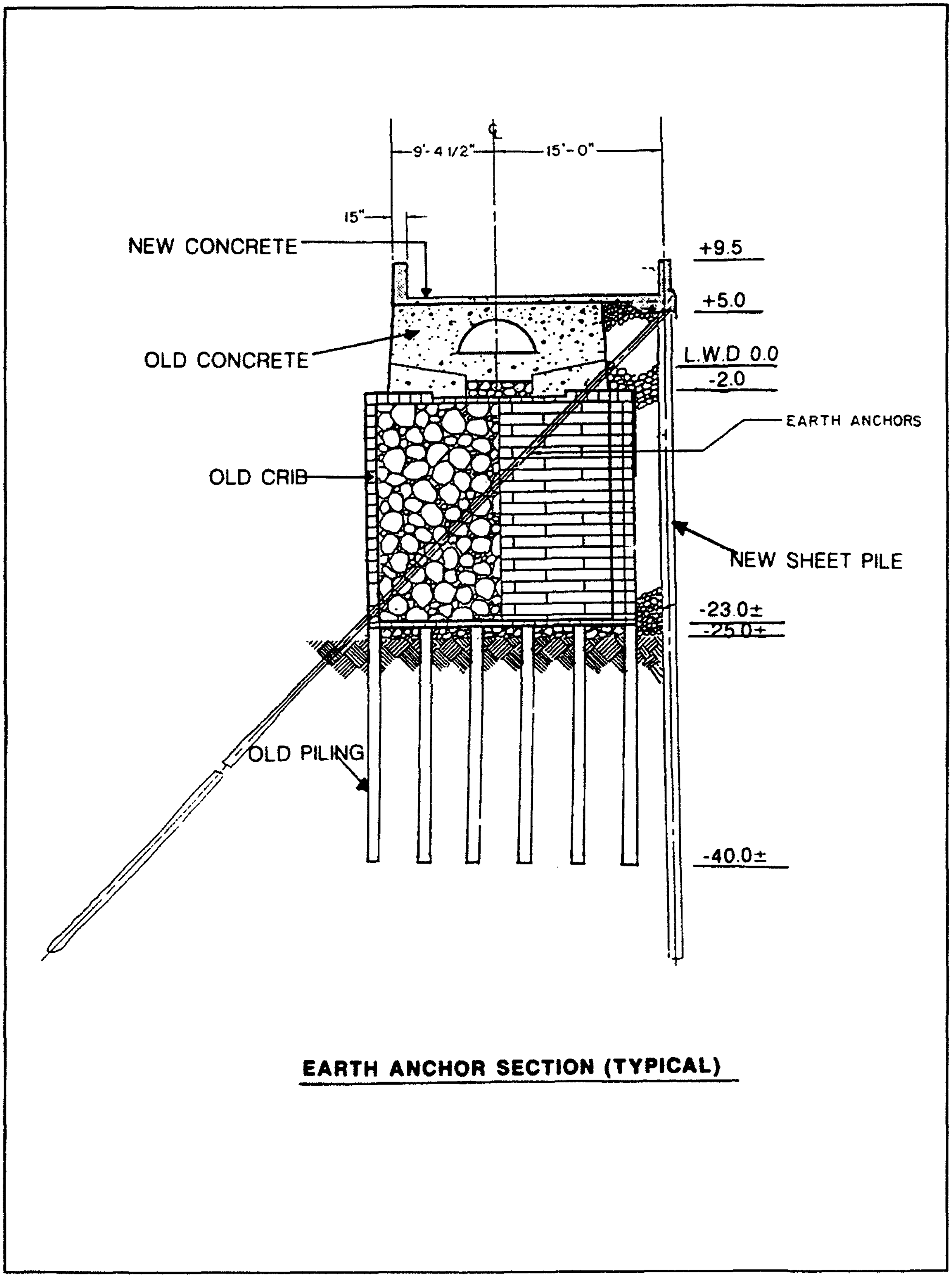

Figure 3. Earth anchor section (typical)

information available about the design and testing of earth anchors. We developed a rational method for design based on the soil stratigraphy and characteristics. Figures 4 and 5 are examples of design computations.

The first step in the design was to determine the required anchor capacity based on the computed sheet-pile anchor pull, the proposed anchor spacing, and the anchor test requirements. The top of the bonded length of the anchor bulb was computed by extending a line upward from the point of fixity of the sheet-pile wall at an angle of $45 - ø/2$[1] deg from vertical. Then, the elevation of the bottom of the anchor was estimated. The diameter of the grout bulb in the bonded length of the anchor was estimated based on the soil characteristics (type, density). Then the adhesive/friction forces between the soil and the grout bulb and the soil were computed and compared with the required anchor capacity. The depth of the anchor was adjusted, if necessary.

The anchor tendon(s) were sized based on the test load. The high-strength steel was stressed up to 0.8 of the guaranteed ultimate tensile strength at the maximum test load. The wale beam was sized based on the design load. Because the anchors penetrated the wale beam, reinforcing for the wale was necessary. Figure 6 shows an example of how a wale was reinforced at anchor locations.

Construction

Earth anchor installation is as much an art as it is a science. It involves many variables such as drilling method, grouting pressure, and tendon type. Anchor installation is regarded as a more complicated and difficult type of construction than "normal" tiebacks using an A-frame, anchor wall, or a deadman. Therefore, in the Detroit District, earth anchors are used where there is some reason that a deadman or anchor wall cannot be installed, such as an existing structure that cannot be economically moved. See Figure 7 for an illustration.

Construction methods for earth anchors vary with different contractors and subsurface conditions, but there is enough standardization to describe a typical installation procedure. The first step is to set up a drill rig at the anchor location. The second is to drill through the existing structure, which might be wood, concrete, stone, or a combination of the three. Next a casing, normally 5 in. or more in diameter, is drilled or driven through the soil to the design depth for the bottom of the anchor. A cement grout is mixed on the surface in a small-scale mixing plant. The grout strength is specified as 3,000 psi after 28 days. The hole is filled with grout using a tremie hose. Then the tendons (cables or a bar) are inserted all the way to the bottom of the hole. A pump and grout line is attached to the top of the casing and pressure is applied to the grout column. At the same time the pressure is applied, the casing is slowly withdrawn, usually in 2-ft increments. The pressure[2] is applied until there is a sudden build-up, then the next increment of casing is withdrawn. This process is continued until the entire bonded length of the anchor is pressure grouted. After the remaining portion of the casing is tremie grouted, the sheath and the "trumpet" are then installed and the anchor is allowed to cure for 7 days. After curing, anchor is tested. If the test is successful, a small posttensioning load is applied to the anchor, and it is permanently connected to the sheet-pile wall.

Testing and Evaluation

As mentioned previously, earth anchors are tested to ensure that they will perform as designed. The test procedures that Detroit District has been specifying were adopted in the early 1970's and were based on the load testing of battered piles. The testing for an

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1 ø is the angle of internal friction of the soil.

2 The maximum pressure is normally about 150 to 300 psi measured at the top of the casing.

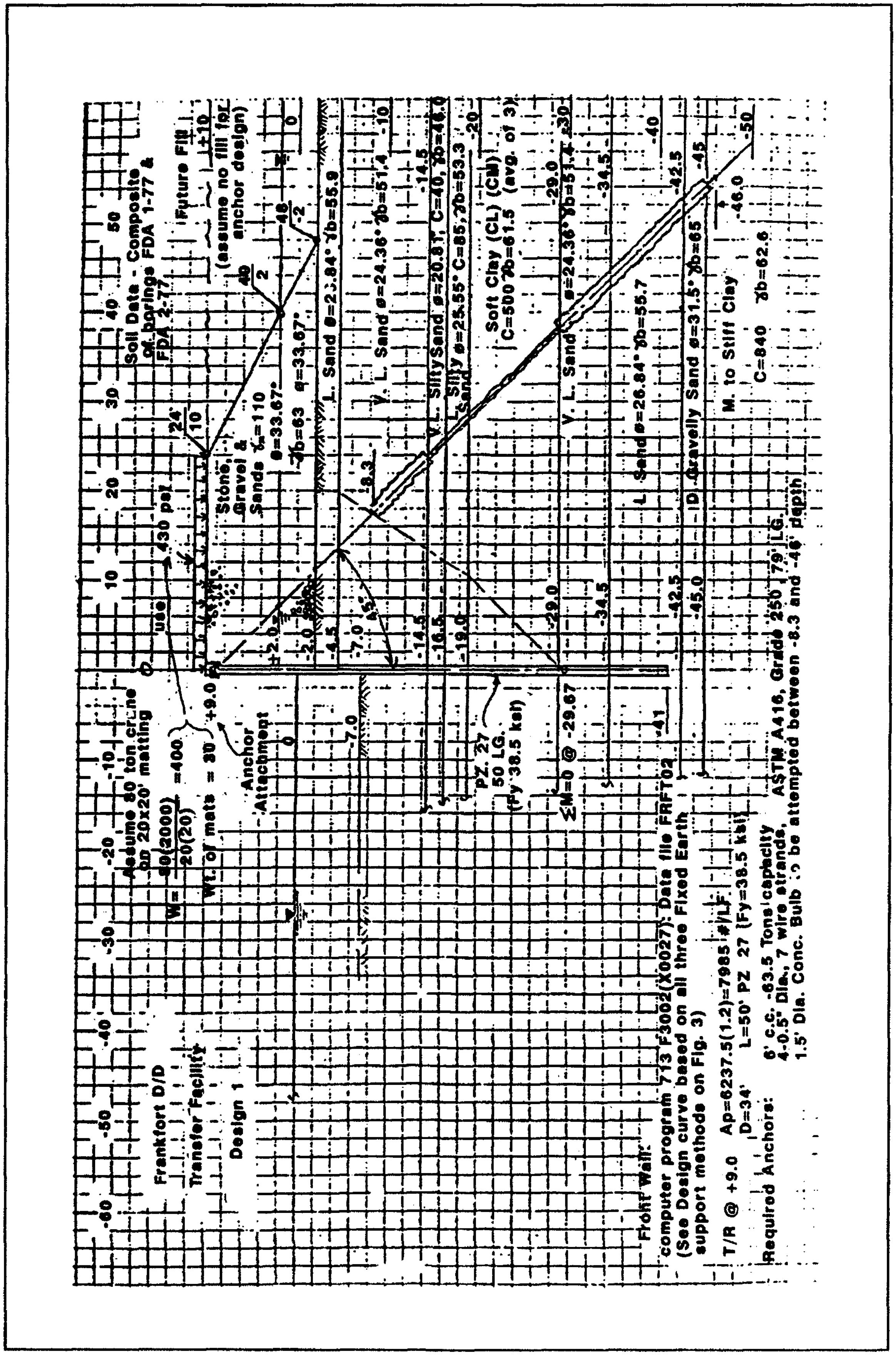


Figure 4. Design computations

Multi-strand Earth Anchors 6'c.c.

$\beta = 45^\circ$, Anchor attachment @ +9.0

Pull-Out force (axial) = $\dfrac{7485(6)(2.00)}{\text{Sin } 45^\circ}$ = 127025# = 63.5 Tons

Vertical Pressures along anchor :

Depth	Geostatic (below -2)	Strip Ldg 2b=24;p=430 z=d+10 x=(z-1)-12	1/2 Trapezoid a=24;B=43 p=12(63)=756 z=d-2 x=37-z	Trapezoid (above +2) a=16;b=40 p=8(47)=376 z=d+2 x=33-z	Composite
-8.3	335.07	264.21	737.72	351.38	1688.4
-14.5	653.75	178.61	629.91	280.03	1742.3
-16.5	745.75	157.79	580.66	253.52	1737.7
-19.0	879.00	136.21	517.90	222.20	1755.3
-29.0	1494.0	83.44	311.38	133.85	2022.7
-34.5	1776.7	67.52	239.52	106.42	2190.2
-42.5	2223.9	52.31	173.11	81.13	2530.5
-45.0	2386.4	48.78	158.54	75.45	2669.2

Resistance to Pull-out

$$R_s\ -8.3 \rightarrow -14.5 = \frac{6.2(1.5\pi)(.625)}{\text{Cos } 45^\circ}\left(\frac{1688.4 + 1742.3}{2}\right)\text{Tan } 24.36^\circ = 20{,}057.2$$

$$R_s\ -14.5 \rightarrow -16.5 = \frac{2.0(4.5\pi)(.625)}{\text{Cos } 45^\circ(12)}\left(\frac{1742.3 + 1737.7}{2}\right)\text{Tan } 20.81^\circ = 1377.2$$

$$R_{coh}\ -14.5 \rightarrow -16.5 = \frac{2.0(4.5\pi)(40)}{\text{Cos } 45^\circ(12)} = 133.2$$

$$R_s\ -16.5 \rightarrow -19 = \frac{2.5(4.5)(\pi)(25)}{\text{Cos } 45^\circ(12)}\left(\frac{1737.7 + 1755.3}{2}\right)\text{Tan } 25.55^\circ = 2168.0$$

$$R_{coh}\ -16.5 \rightarrow -19 = \frac{2.5(4.5)(\pi)(25)}{\text{Cos } 45^\circ(12)} = 104.0$$

$$R_{coh}\ -19 \rightarrow -29 = \frac{10.0(4.5\pi)(500)}{\text{Cos } 45^\circ(12)} = 8330.0$$

$$R_s\ -29 \rightarrow -34.5 = \frac{5.5(1.5\pi)(.625)}{\text{Cos } 45^\circ}\left(\frac{2022.7 + 2190.2}{2}\right)\text{Tan } 24.36^\circ = 21849.0$$

$$R_s\ -34.5 \rightarrow -42.5 = \frac{8.0(1.5\pi)(.625)}{\text{Cos } 45^\circ}\left(\frac{2190.2 + 2530.5}{2}\right)\text{Tan } 26.84^\circ = 39798.0$$

$$R_s\ -42.5 \rightarrow -45 = \frac{2.5(1.5\pi)(.625)}{\text{Cos } 45^\circ}\left(\frac{2530.5 + 2669.2}{2}\right)\text{Tan } 31.5^\circ = 16589.0$$

(110,408.2)

$$Q_s\text{-}29.0 = (cN_c+\sigma_z)(\pi/4)(d_b^2-d_s^2) = (500(9)+2022.7)(\pi/4)(1.5^2-(4.5''/12)^2) = 10{,}808.2$$

121,214.4

Wt. of Conc. Bulb (-8.3 -45): $L_b = \dfrac{(45-29)+(14.5-8.3)}{\text{Cos } 45^\circ} = 31.4'$

$$\text{Wt.} = \frac{(1.5)^2}{4}\pi(31.4)(150-62.4) = 4{,}860.8$$

Wt. of Conc. Shaft (-14.5→-29): $L_s = \dfrac{23-14.5}{\text{Cos } 45^\circ} = 20.5'$

$$\text{Wt.} = (4.5/12)^2(\pi/4)(20.5)(150-62.4) = 198.3$$

126,273.5

D = added depth below = -45:

$$127025 = \frac{126273.5 + D(4.5/12)\pi(840)}{\text{Cos } 45^\circ} \qquad D=0.54'$$

Anchor Tip @ -45.54, Say -46

$L_{cable} = \dfrac{9+46}{\text{Cos } 45^\circ} = 77.78 + 1.22 = 79'$

Multi-Strands: ASTM A416, grade 250

Try 4-0.5" Dia 7-wire strands (A_s=.153 sq. in.)

Avail. Capacity = 4(.153)(250000) = 153,000 > 127,025 o.k.

Figure 5. Design computations

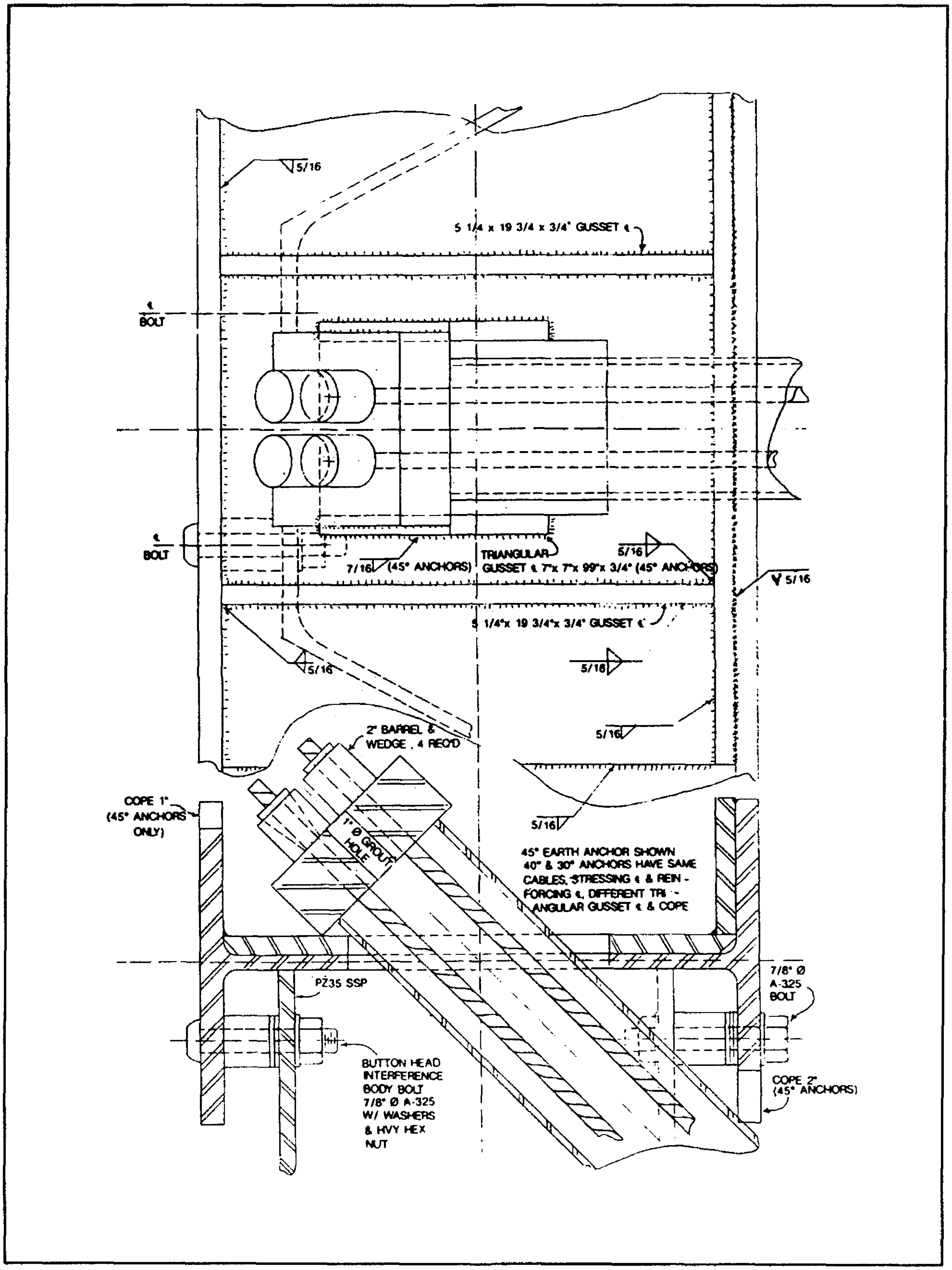


Figure 6. Wale reinforcement at anchor locations

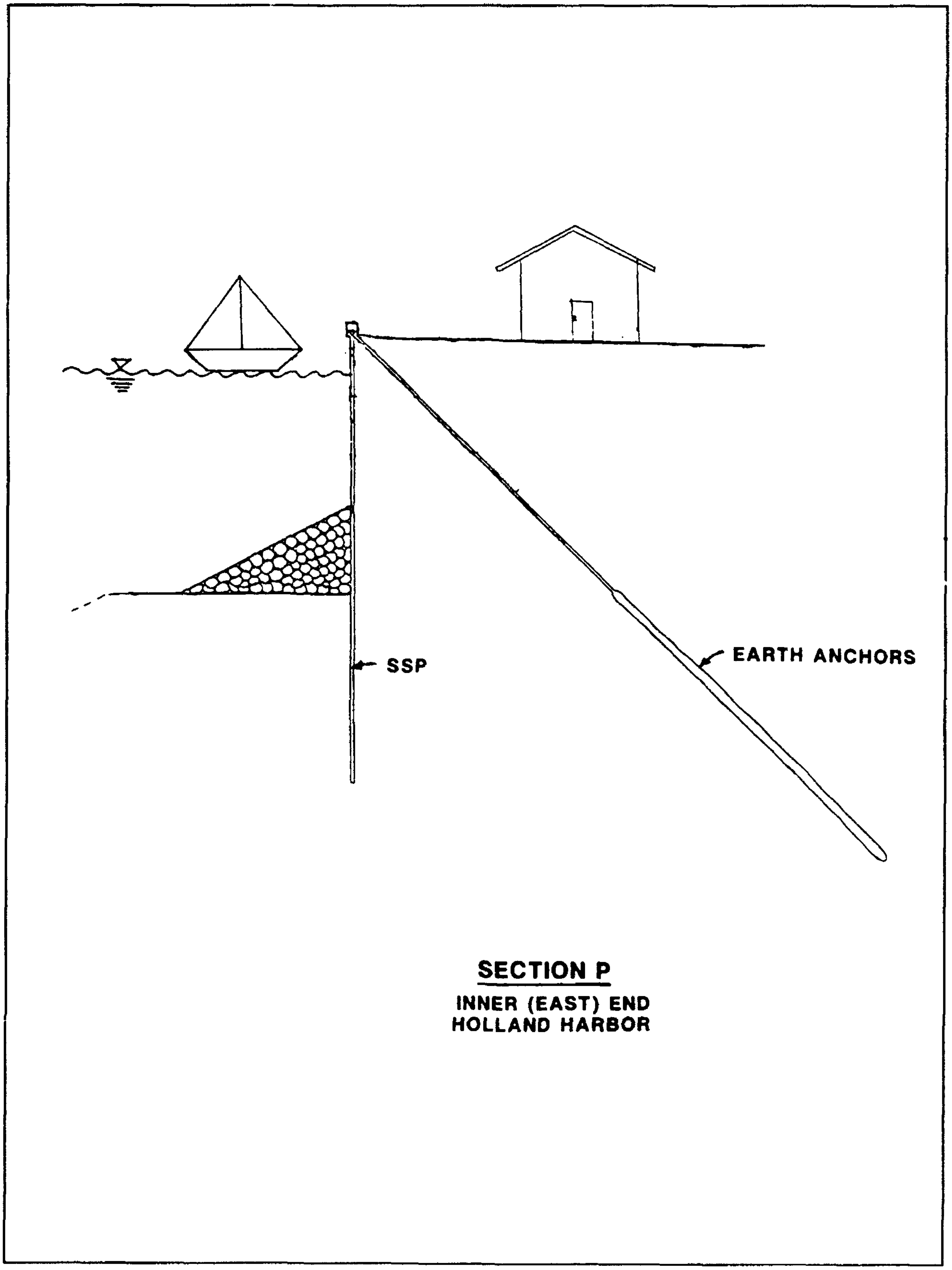


Figure 7. Existing structure requiring earth anchor

earth anchor job consisted of two types of tests, performance, and proof. Test loads were applied by a hydraulic jack and movement of the tendons was monitored by dial gages.

The purpose of the performance tests was to verify the design of the anchors. Two or more anchors (the number depending on the size of the job and the uniformity of the subsurface conditions) were tested at the beginning of the job. If the anchors met the test criteria, the contractor proceeded with the installation of production anchors. If the anchors did not meet the testing criteria and were installed correctly by the contractor, they were redesigned at Government expense.

The performance test criteria adopted by the Detroit District in the early 1970's were fairly severe. The anchors were tested to two times the sheet-pile anchor pull design load. Since the wale beam was not designed for the test load, a special reaction frame was required. We required that the special reaction frame be able to sustain three times the test load (six times design load). The test load was cycled up to the maximum and then held at maximum for 24+ hr to test for creep (Figure 8). Failure criteria was a net movement of less than 0.0075" per ton of test load.

Proof testing criteria was similar. The purpose of the proof testing was to verify that the production anchors were acceptable. The Corps selected 10 percent of the production anchors for proof testing. The anchors were tested to twice the design load. The same reaction frame was used and the failure criteria was the same as the performance test. The loading was not cyclical and the maximum load was held for 1+ hr.

Recent Detroit District Earth Anchor Projects

After numerous successful projects in the 1970's and early 1980's the district had problems with three successive earth anchor projects. The first was the mid-1980's in Grand Haven, MI, where we rehabilitated a section of revetment along the Grand River near the river's entry into Lake Michigan. The contractor had difficulty installing the initial performance test anchors, and the district had to redesign them. Because of this we had to pay a substantial construction claim.

The second project was in Duluth, MN, about 2 years after the Grand Haven project. In this project we rehabilitated revetments along both sides of the Duluth Ship Canal. There were more than 100 anchors on each side of the canal and different subcontractors installed the anchors on each side. They both filed substantial construction claims which have been denied and are currently being appealed.

The third project was constructed at Holland, MI. At Holland, the Black River empties into Lake Macatawa, and Lake Macatawa empties into Lake Michigan through a channel about 1/3 mile long. The Corps maintains the channel between Lake Macatawa and Lake Michigan. In this project, we rehabilitated the south side of the channel. Where there was enough land available we replaced a deteriorated Wakefield sheet-pile wall with a stone revetment. The eastern end of the channel narrows and it was necessary to replace the wooden pile wall with a sheet-steel pile wall. This section of the project is designated "Section P " and is shown in Figure 9.

There are several cottages very near the channel in Section P, which eliminated the possibility of a conventional tieback wall. Therefore, we decided to use earth anchor tie backs for the new sheet-pile wall. Because of real estate problems, Section P was divided into two segments. The first segment was the western part of Section P and was designed and constructed in the late 1980's.

When the contractor installed the performance test anchors in the western segment of Section P, the anchors did not meet the acceptance criteria. After this happened, there was considerable discussion between the Corps and the contractor as to the reason the anchors did not meet the specified criteria. The contractor claimed that the anchors were good

and the testing criteria was too severe. The Corps questioned whether the contractor had pumped an adequate amount of grout into the bonded length of the anchors. After much discussion and some additional soil borings, we decided to add 10 ft to the length of the anchors and require the contractor to use higher grouting pressure to place more grout and form larger grout bulbs. The longer anchors successfully met both performance and proof testing requirements. However, a substantial construction claim resulted due to delays and the necessity for the contractor to purchase longer tendons.

Reevaluation of District's Earth Anchor Criteria

After the completion of the western segment of Section P at Holland Harbor, a decision was made by the Chiefs of Design Branch and Geotech Branch to review the district's earth anchor criteria before building the eastern segment. The decision was based on input from the Construction Branch and Project Offices, experience on recent projects, and review of recent technical literature.

Field construction personnel had reservations about additional earth anchor projects based on the fact that the last three had all resulted in substantial construction claims. All three had taken longer than scheduled, and all three contracts had been difficult to administer because of disputes with contractors related to earth anchors. Field personnel also objected to the cumbersome testing criteria, particularly the performance tests, which usually took 30 or more hours and had to be closely and continuously monitored.

The available technical literature about earth anchors included ASCE Geotechnical Special Publication No. 4, Tiebacks for Bulkheads (McMahon 1986). This book contains experience and recommendations of several engineers and contractors. Another valuable publication was a Federal Highway Administration publication titled Permanent Ground Anchors (Cheney 1988). Earth anchors have been used extensively to stabilize highway cuts and the publication summarizes this experience. It includes recommendations for design, construction, and testing of anchors. Both of these publications recommended design and testing criteria and procedures different from those the Detroit District had been using for more than 15 years.

Implementation of New Criteria

For the reasons outlined in the previous section, the Chiefs of Design Branch and Construction Branch decided to develop new earth anchor criteria before designing the remainder of Section P at Holland Harbor. The new criteria were developed by William Lawhead,[1] Structural Design Section, Darrel Suhre, Cost Estimating and Specifications Branch, and Ronald Erickson of Geotechnical Engineering Section, Detroit District. The new criteria were based on the individuals' extensive experience and the two publications cited. The new criteria were incorporated into revised earth anchor specifications. The specification changes were primarily in the area of testing requirements. However, since anchor depth and tendon size designs are based on the test load, the new criteria produced less conservative and more constructable anchor designs.

The new (for Detroit District) performance testing requirements include a lower maximum load (1.5 times design load versus 2.0 times design load previously). The loads are held for much shorter intervals, and there are fewer steps to get to the maximum load. The requirements for the test frame are also less severe (See Figure 8 for a comparison of old and new test methods). The failure criteria is also changed from net movement during the test to total movement during the time the maximum load is applied. There are additional criteria to ensure load is being transferred to the bearing strata.[2]

[1] Currently, Mr. Lawhead is project manager for Holland Harbor in Engineering Management Branch.

The most significant change in the proof test procedure is that all anchors are now proof tested instead of just 10 percent of them. The maximum test load is much smaller (1.1 times design load versus 2.0 times). The use of a reaction frame is no longer necessary due to the lower loading. For the proof tests, there are also changes in loading intervals and failure criteria similar to the changes in the performance test.

Other discussions that took place during the reevaluation of earth anchor design criteria dealt with whether the contractor should be allowed to select his own method of anchor installation or whether the specifications should be the "cookbook" type with construction procedures outlined. The new specification gives some guidelines but allows the contractor to select his method of drilling and grouting.

Another possibility that was discussed was specifying an anchor pull and allowing the contractor to select the anchor depth. We decided to continue specifying the drilling angle, a depth for the top of the grout bulb, and a minimum depth for the bottom of the bulb.

Conclusions

The design for the remaining segment of Holland Harbor has been completed using the district's new earth anchor criteria. Figure 10 compares the old and new designs. Note that the anchors for the more recent design are 13 ft shallower.

The contract has been awarded and construction is underway. As of June 1991, the two required performance tests have been successfully completed. Production anchors are being installed and the first proof tests have also been successful.

Based on our recent experiences, earth anchors are a viable method for steel sheet-pile tiebacks and should be considered for many designs. We learned that Corps procedures, customs, and practices should be reviewed periodically and compared with the standard practices of other agencies and private industry. Any major differences should be evaluated and, if necessary, Corps procedures should be changed.

Another conclusion is that the observations of field construction personnel should be carefully considered and, if possible, incorporated into future work.

Acknowledgement

Several people provided information for this paper. We would like to thank William Lawhead, Darrel Suhre, Thomas O'Bryan, James Schulz, Lawrence Moloney, and David Foster, Detroit District, and Hari Singh and Joseph Schmidt, North Central Division. Sheetal Patel, Detroit District, prepared the illustrations and assembled the paper.

References

McMahon, Donald R. 1986. Tiebacks for Bulkheads, ASCE Geotechnical Special Publication No. 4.

Cheney, Richard S. 1988. Permanent Ground Anchors, Federal Highway Administration Report FWHA-DP-68-1R.

[4] Total movement at maximum load must exceed 0.8 times (PL/AE) for tendons, where L is the length from the top of the cable to the center of resistance of the grout bulb.

COMPARISON OF EARTH ANCHOR SPECIFICATIONS
40 TON DESIGN LOAD (EXAMPLE)

NEW SPEC.	OLD SPECS
I. Performance (initial) test	I. Performance (initial) test
a. 1.5 x design load = 60 tons	a. 2.0 x design load = 80 tons
b. reaction frame: 3 x design load (example, 40 tons x 3 = 120 tons)	b. reaction frame: 3 x test load (example, 40 tons x 2 x 3 = 240 tons)
c. loading sequence: 0, A*, 10 tons, 20 tons, A, 20, 30, 20, A, 20, 30, 40, 20, A, 20, 30, 40, 50, 60 tons, A	c. loading sequence: 0, A, 10 tons, 20, 30, 20, 10, A, 10, 20, 30, 40, 50, 60, 50, 40, 30, 20, 10, A, 20, 30, 40, 50, 60, 70, 80 tons, 70, 60, 50, 40, 30, 20, 10, A
d. hold: 1 minute @ each load	d. hold: 1 hour to 2 hours @ each load
e. creep: 100 minutes	e. creep: 24 hours +
f. pass: movement < 0.08" during interval 10 minutes to 100 minutes or < 0.04" during 1 min. to 10 min.	f. pass: net movement @ end of test < 0.0075" per ton of test load
II. Proof (production) test	II. Proof (production) test
a. every anchor	a. 10% of anchors
b. 1.1 x design load	b. 2.0 x design load
c. reaction frame: may use back-filled ssp wall	c. reaction frame: 3 x test load (example, 40 tons x 2 x 3 = 240 tons)
d. loading sequence: 0, A*, 10 tons, 20, 30, 40, 44 tons, A	d. loading sequence: 0, A, 10 tons, 20, 20, 30, 40, 50, 60, 70, 80 tons, 70, 60, 50, 40, 30, 20, 10, A
e. hold: 1 minute @ each load	e. hold: 1 hour - loading & unloading
f. e. creep: 10 or 100 minutes	f. creep: 1 hour
g. pass: movement < 0.08" during either 1 minute to 10 minutes or 10 minutes to 100	g. pass: net movement @ end of test < 0.0075" per ton of test load

* A small alignment load is applied prior to the test. This is designated "A".

Figure 8. Comparison of earth anchor specifications, 40-ton design load (example)

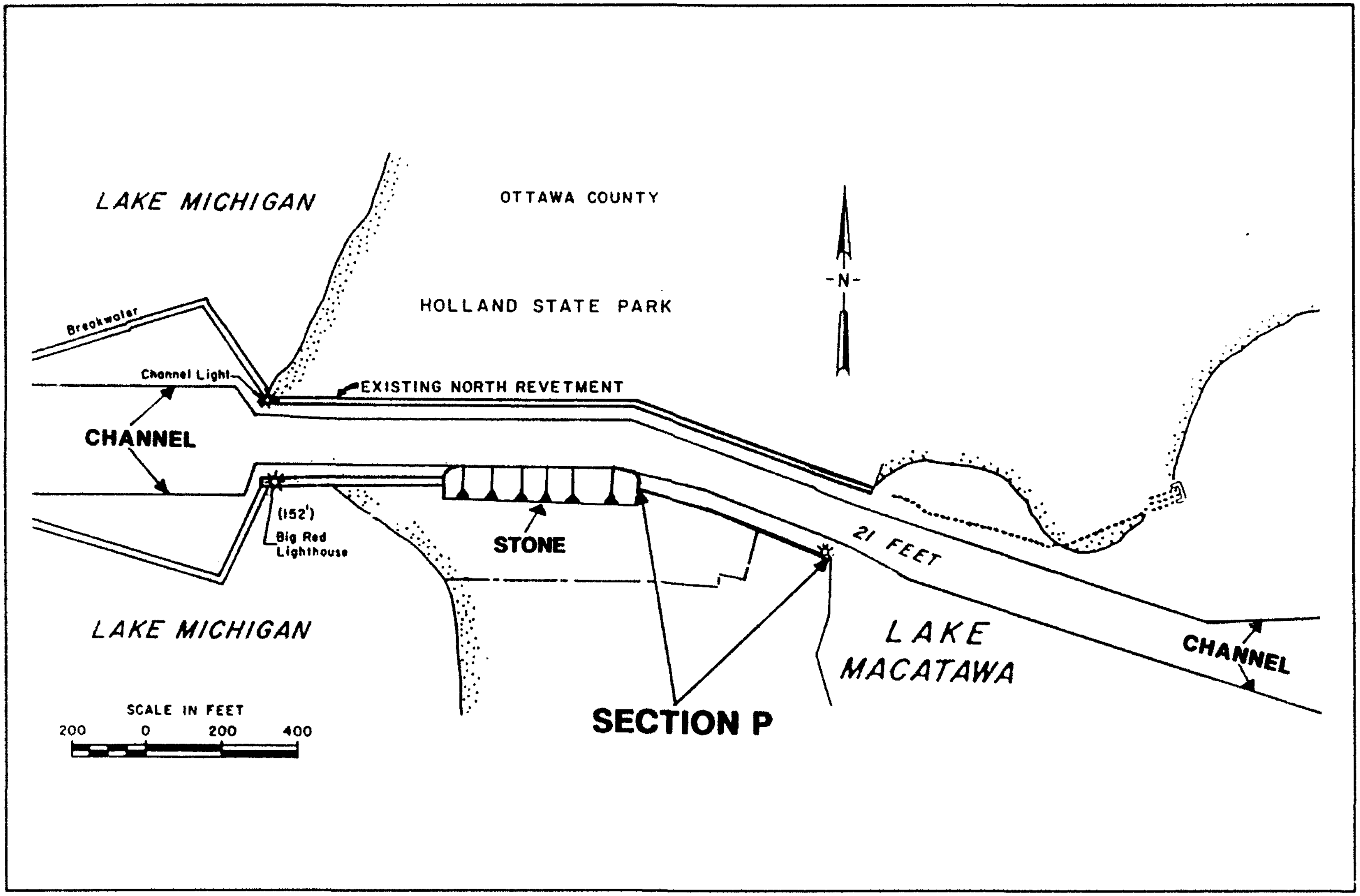


Figure 9. Section of earth anchor project requiring steel sheet-pile wall

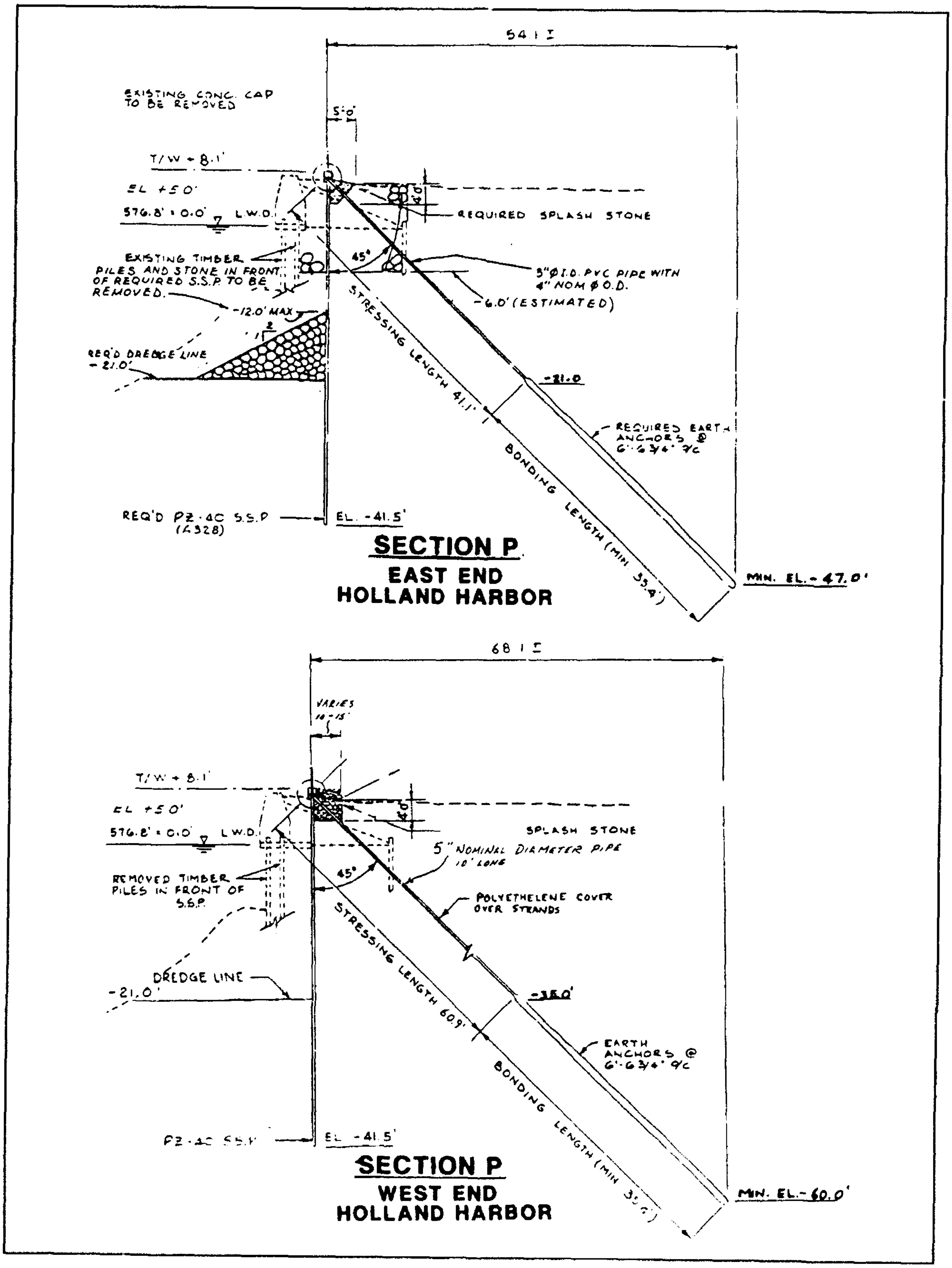


Figure 10. Comparison of Holland Harbor new and old designs

Temporary Tieback Wall, Bonneville Navigation Lock

by
Dale F. Munger,[1] Patrick T. Jones,[2] and Joseph Johnson[3]

Abstract

During construction of the second navigation lock at Bonneville Lock and Dam, a 50-ft-high (15.2m) temporary tieback reinforced concrete diaphragm wall was built to retain the foundation of the Union Pacific Railroad. Wall deflections were a great concern; therefore, a soil structure interaction beam-column analysis was performed to predict wall movements. The wall was heavily instrumented to monitor deflections, moments, and tendon loads. Parameter studies were conducted to calibrate the calculated deflections and moments with the measured performance of the wall. The initial predictions of the deflection and moments were reasonably close to the measured values. Increasing the subgrade modulus brought the calculated deflections closer to the measured deflection, but did not significantly change the magnitude of the calculated moments.

Introduction

A temporary tieback wall was instrumented to monitor its performance during construction of the Bonneville Second Navigation Lock and to provide data to guide the design of future tieback walls at the project. Soil-structure interaction beam-column analyses using nonlinear Winkler spring supports were conducted to predict the performance of the temporary wall. After construction, a parametric study of the model was conducted to match the model's performance with the observed behavior.

Project Description and Layout

Bonneville Lock and Dam is located on the Columbia River 42 miles east of Portland, OR (Figure 1). The new navigation lock is being constructed between the existing lock on the north and the Union Pacific Railroad's transcontinental rail line on the south (Figure 2).

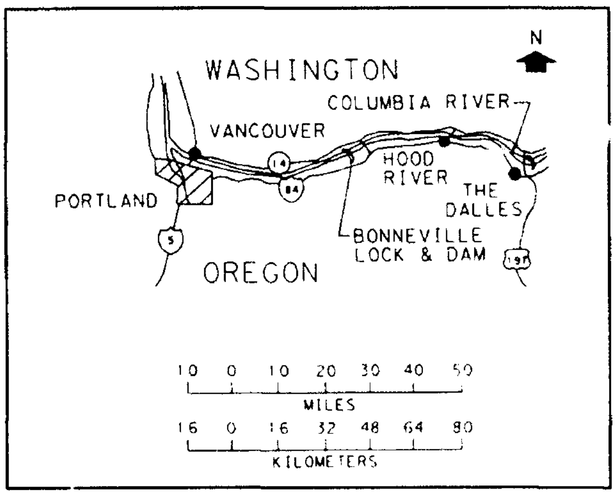


Figure 1. Vicinity map

[1] Geotechnical Engineer, US Army Engineer Division, North Pacific; Portland, OR.
[2] Geotechnical Engineer, US Army Engineer Division, North Pacific; Portland, OR.
[3] Structural Engineer, US Army Engineer Division, North Pacific; Portland, OR.

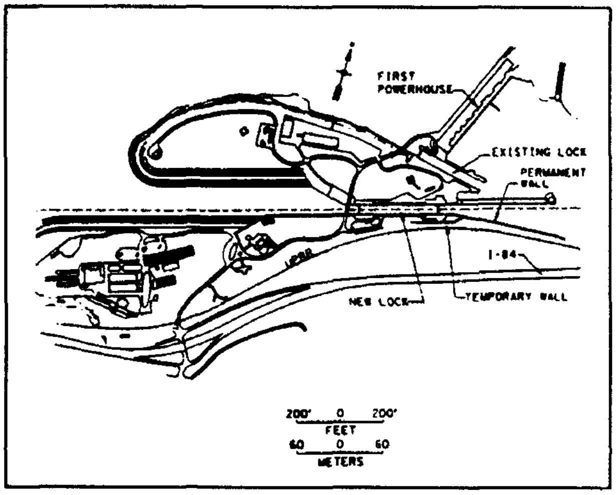


Figure 2. Site plan

Because there is no room for normal open-cut excavation slopes, 180 lin ft (54.9m) of temporary tieback wall and 970 lin ft (295.7m) of permanent tieback walls have been or will be constructed for the new navigation lock. The total wall areas are 3,600 sq ft (334.5 square meters) and 108,000 sq ft (10,034 square meters), respectively.

The temporary tieback wall was completed in February 1988 as part of the lock chamber excavation contract. The lock chamber was excavated into a diabase intrusive, which extends up and downstream just enough to accommodate the main lock structure. At the upstream end of the lock chamber, the top of rock drops off rapidly, requiring installation of a temporary tieback wall to retain the soil foundation of the adjacent railroad. The maximum height of temporary tieback wall exposed during the initial lock chamber excavation contract was 48 ft (14.6m) and the exposed height of wall in future contracts will exceed 100 ft (30.48m). The railroad is only 45 to 50 ft (13.7 to 15.2m) away from the edge of the excavation.

Site Characterization

Extensive exploration, sampling, and testing were performed to characterize the site. Site geology is a result of massive ancient landslides, regional volcanics, and alluvial deposits from large floods. The upper zone of material to be retained by the temporary and permanent tieback walls is Reworked Slide Debris (RSD). RSD is a heterogeneous mixture of alluvial silts, sands, gravels, cobbles, and boulders mixed with angular rock fragments of igneous and sedimentary origin from old landslide masses. Underlying the RSD is the Weigle Formation, which is composed of fine-grained, volcanic-derived sedimentary rocks.

The Weigle formation consists of interbedded mudstones, sandstones, siltstones, and claystones. In the area of the temporary wall the Weigle Formation is underlain by the Bonney Rock diabase intrusive. The diabase bedrock is a basalt-like rock with a columnar structure (Figure 3).

The natural groundwater regime at the wall site is dominated by the Bonneville pool with a smaller influence from hillside drainage. Dewatering was accomplished to top of rock behind the wall using wells.

Material Properties

The friction angle and cohesion values given in Table 1 are lower bound values for the RSD and Weigle Formation and are based on laboratory and in-situ testing. Samples were obtained by core and cable-tool drilling and from test pits. Densities were obtained from core samples and block samples from the test pits. Strength data were obtained from direct and triaxial shear testing. The constant of horizontal subgrade reaction (L_h) was selected from values experimentally derived by Terzaghi (1955) for retaining walls embedded in soil. For the purposes of choosing L_h, the RSD and Weigle Formation materials were assumed to be cohesionless. The selected L_h values were weighted toward the higher end of the Terzaghi values to reflect the relatively high unit weights of the materials. The RSD was assumed to behave as a medium-dense sand and the Weigle Formation was assumed to behave as a "soft" sedimentary rock. The value of L_h changes with submergence for the RSD because it is a soil, while the Weigle

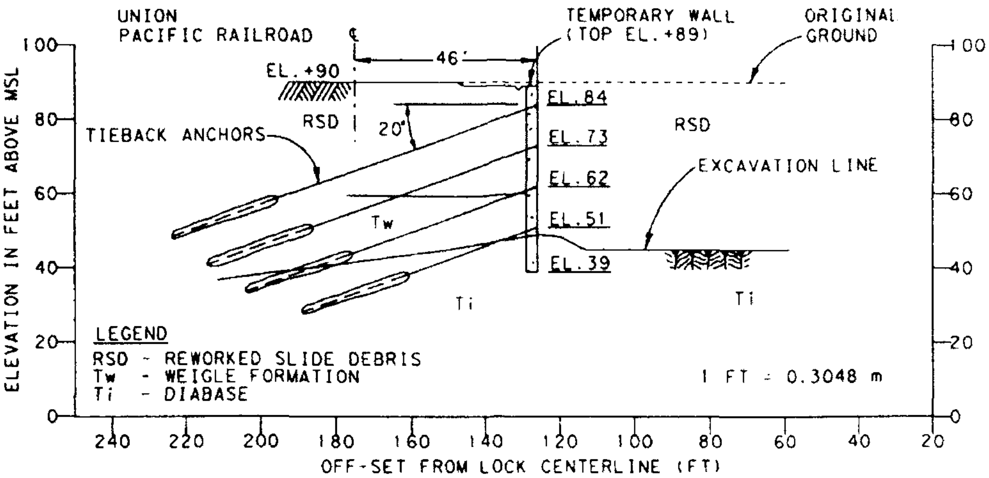


Figure 3. Descriptive cross section

formation L_h remains constant whether dry, moist, or submerged because it is essentially a weak sedimentary rock.

Test Anchor Program

The geometry of the geologic formations at the site dictated that most of the tieback anchors would have bonded zones in soil materials with a few in rock (diabase). The proposed use of permanent soil anchors and the unknown strength capacity of these formations were the reasons for conducting large scale anchor tests at the site in the summer of 1986. The anchor tests confirmed that high capacity soil anchors could be installed at the site using "tremie" grouting methods. Tremie grouting is the pumping of the grout mixture through a tube, which is installed with the tendon so that the hole is filled with grout from the bottom up to the surface. No additional pressure is used above that of the hydrostatic pressure of the grout column. Tremie grouting was used for the test anchors to obtain minimum baseline data for anchorage capabilities of materials at this site. The allowable bond stresses obtained for the RSD and Weigle Formations were 60 psi (413.7kN/m^2) and 35 psi (241.3kN/m^2), respectively. The tests showed that long-term creep would not be a problem at the proposed design loads. The measured creep rates were less than 0.04 in. (1.02 mm) per log cycle for five of the six test anchors. These rates were significantly less than the allowable Post Tensioning Institute (1986) standard of 0.08 in. (2.03 mm) per log cycle.

Table 1
Material Properties

Parameter	**RSD**	**Weigle Formation**
Friction angle (deg)	30	30
Cohesion (psi)[1]	0	10
Moist unit weight (pcf)[2]	125[4]/118[5]	142.5[4]/130[5]
Saturated unit weight (pcf)	130	142.5
Constant of horizontal subgrade reaction L_h		
Moist (pci)[3]	18	23
Submerged (pci)	13	23

[1] 1 pound (force) per square inch = 6.9 kilonewtons per square meter.
[2] 1 pound per cubic foot (denisty) = 1.6 kilograms per cubic meter.
[3] 1 pound (force) per cubic inch = 271.45 12 kilonewtons per cubic meter.
[4] Unit weight used to develop earth pressure distribution for input into CFRAME or analysis.
[5] Unit weight used in CBEAMC analysis, changed due to accumulation of additional in-place density data.

Design Criteria

To prevent settlement of the railroad foundation and to eliminate the possibility of initiating a landslide in this highly unstable area, the maximum allowable horizontal wall deflection under normal loads was set at a conservative 0.1 ft (30.5mm) riverward anywhere along the height of the wall. A concrete diaphragm wall, constructed by the slurry trench method, was chosen because of the ability of these walls to resist deflections. The unbonded lengths of the tendons were set so the bonded zones were beyond the design failure surfaces. The design failure surfaces were determined using slope stability limit equalibrium procedures with a factor of safety equal to 1.5 for static load conditions and 1.0 for dewatering system failed and seismic load conditions. The tieback forces were not considered in the analysis to determine the location of the design failure surfaces. A factor of safety of 2.0 was applied to the ultimate grout to soil bond stress to determine required bonded lengths. The tendons were sized using ACI 318-83 with one exception to section 18.5.1(c), which was that the tendon loads were not to exceed 75 percent, instead of the normal 80 percent, of the ultimate tensile strength during anchor testing. Maximum performance and proof testing loads were 1.5 times the design load. Reinforced concrete was designed using ACI 318-83.

Soil Load

A combination of rectangular and triangular at-rest soil loadings, plus a surcharge, was used for wall design (Figure 4). A rectangular apparent earth pressure diagram for tiedback (or internally braced) walls in sand was used to account for the typical load distribution that occurs in these walls. The at-rest rectangular pressure distribution used was taken from Figure 29, Chapter 3 of NAVFAC DM-7.2 (1982). A triangular diagram was used to provide for the possibility that upper anchors could lose some tension during the life of the wall, leading to greater loads at the base of the wall. The at-rest condition was used to limit deformations to a minimum, because the railroad is just adjacent to the top of the wall. Jaky (1944) equation was used to determine a suitable value of K_o.

$$K_o = 1 - \sin\phi \tag{1}$$

where K_o = at-rest stress ratio and ϕ = friction angle. The at-rest triangular soil pressure was calculated using:

$$P_o = K_o \gamma_m H \tag{2}$$

where γ_m = moist unit weight and H = wall height. The ordinate of the rectangular apparent earth pressure diagram was calculated using:

$$P_o = 0.5 K_o \gamma_m H + S \tag{3}$$

with S = surcharge load. Using elastic theory, the horizontal pressures were increased by 0.44 ksf because of surcharge loads from trains and construction equipment. The site was dewatered; thus, no hydrostatic loads were considered. Panel 6 (Figure 5) was designed using a wall height of 54 ft (16.4m). Due to higher than anticipated bedrock the constructed height ended up being 48 ft (14.6m). The design earth pressure distribution was developed using the assumption that the material behind the wall consisted entirely of RSD. Later explorations and slurry trenching indicated the presence of Weigle formation beneath the RSD and above the diabase.

Wall Design

The wall reinforcement (Figure 6) was designed using "CFRAME" (Hartman and Jobst 1983), to determine reactions, moments, and shears. The moments were used to size the vertical reinforcement. CFRAME is a general-purpose computer program for analysis of small or medium plane frame structures. CFRAME has the capability of placing any load on the structure and can generate member shears, moments, and deflections. The wall was modeled vertically as a continuous beam with supports approximately 11 ft apart at each anchor location. This beam was

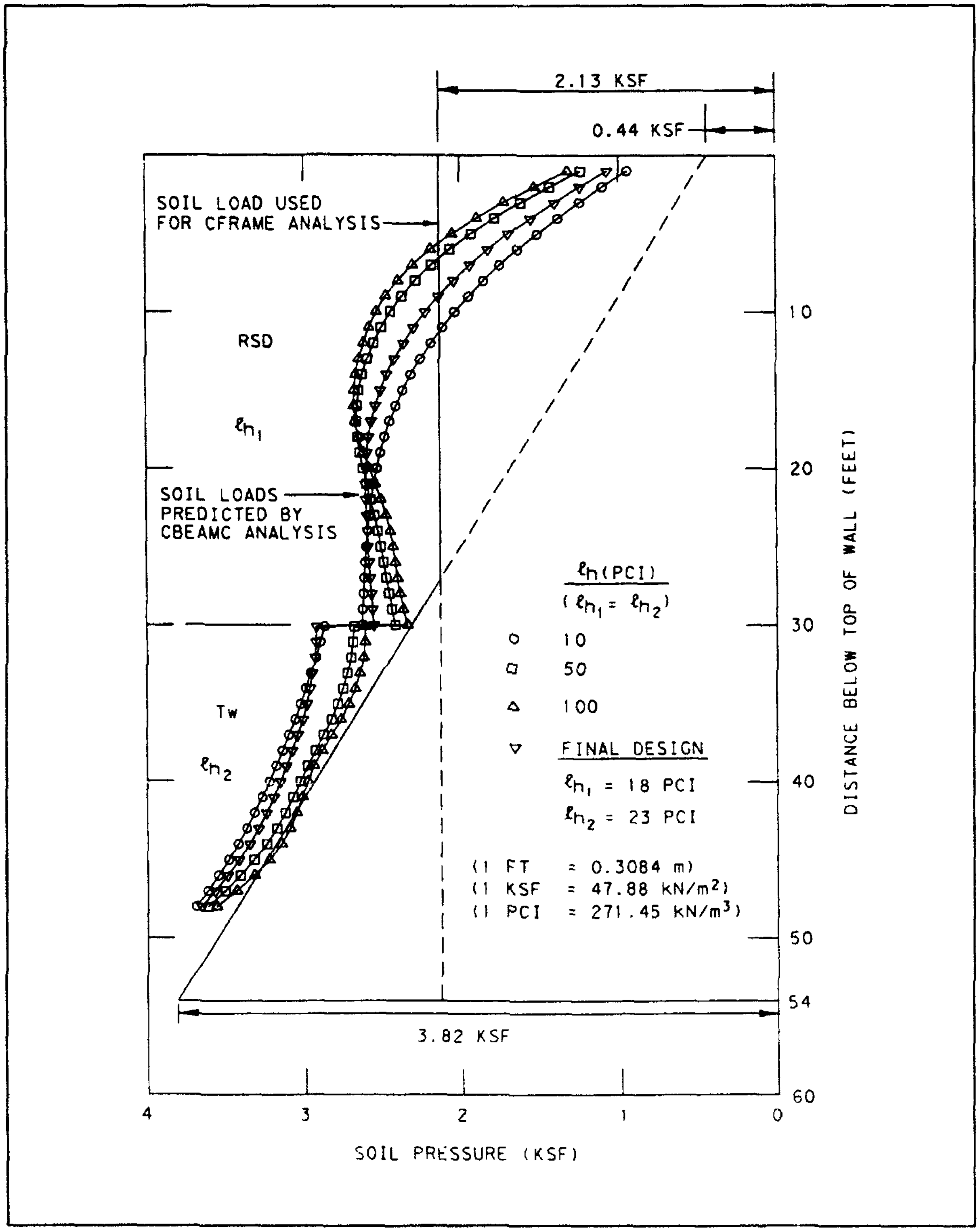


Figure 4. Parametric study of soil pressure

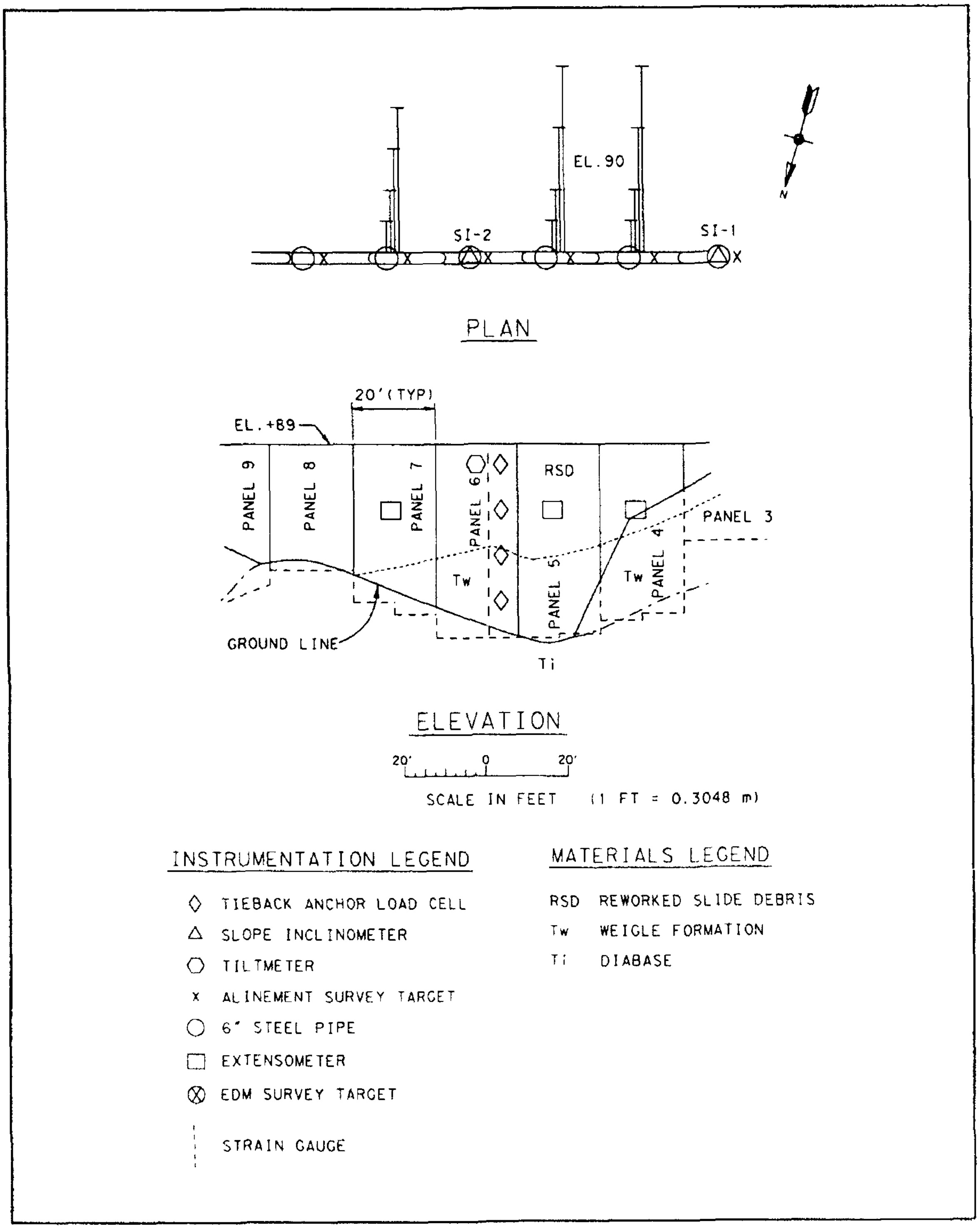
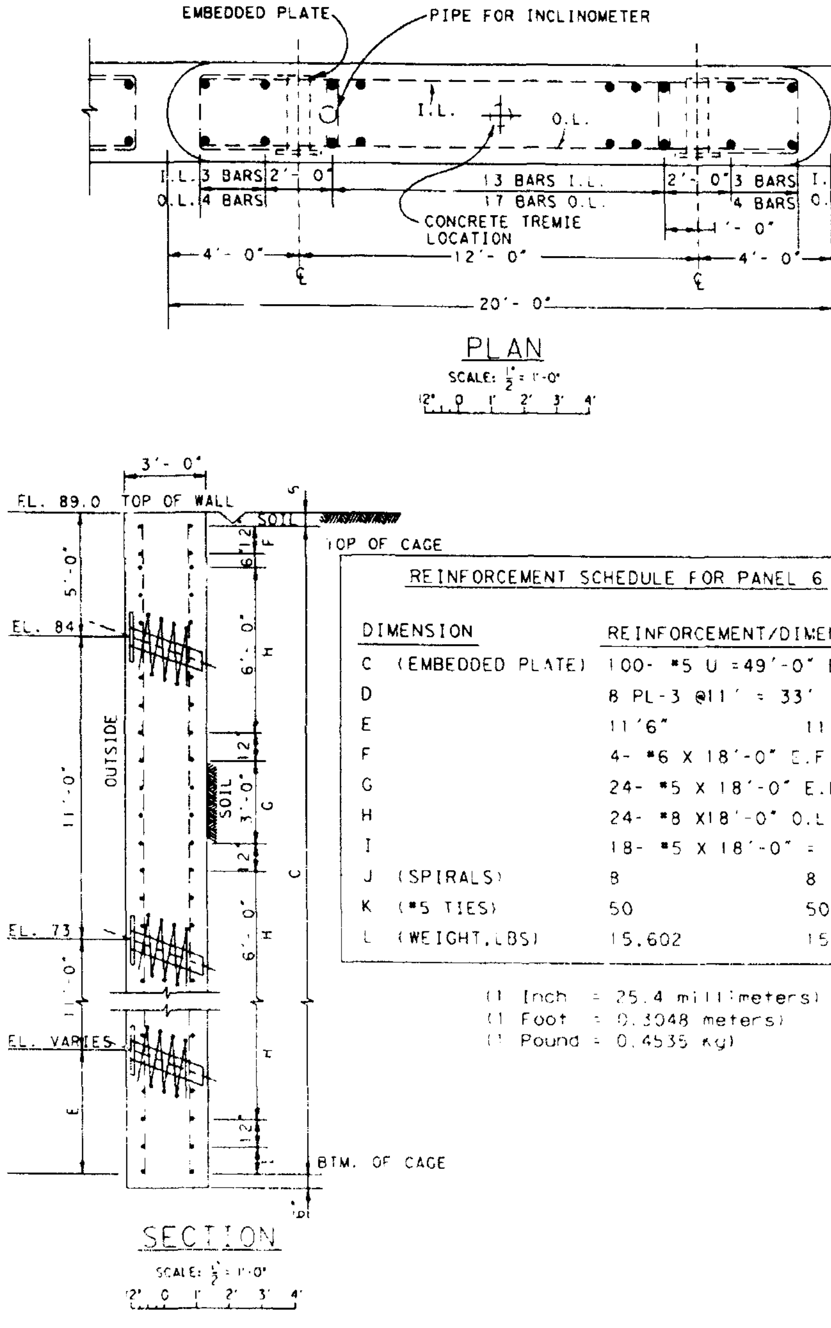


Figure 5. Temporary wall plan and elevation



REINFORCEMENT SCHEDULE FOR PANEL 6

DIMENSION	REINFORCEMENT/DIMENSION	
C (EMBEDDED PLATE)	100- #5 U =49'-0" E.F.	
D	8 PL-3 @11' = 33'	
E	11'6"	11'6"
F	4- #6 X 18'-0" E.F.	
G	24- #5 X 18'-0" E.L. @ 3 PLACES	
H	24- #8 X18'-0" O.L. @ 4 PLACES	
I	18- #5 X 18'-0" = 8'-6" E.F.	
J (SPIRALS)	8	8
K (#5 TIES)	50	50
L (WEIGHT, LBS)	15,602	15,602

(1 Inch = 25.4 millimeters)
(1 Foot = 0.3048 meters)
(1 Pound = 0.4535 kg)

Figure 6. Wall reinforcement

loaded with the earth pressure shown in Figure 4. A horizontal beam was designed to span across the anchor which was loaded with the reaction from the vertical beam. The reaction at each support was used to determine the uniform loading on assumed horizontal built-in beams. The horizontal beam was modeled as a 20-ft (6.1m) beam with a 4-ft (1.22m) overhangs over each support yielding reactions, deflections, moments, and shears. These moments were used to size the horizontal reinforcement. The horizontal component of the tieback design load was taken as the larger of 1 (the reactions from the horizontal beams or 2) the apparent soil pressure over the tributary area of each anchor. There was very little difference between the CFRAME reactions and the loads from the tributary area method. Charts (Figure 7) were developed which show the unbonded length of the tendon versus panel height for the various tendon elevations on the panels. These charts were based on data derived from the limit equilibrium slope stability analysis of three different panel heights. Spencer's procedure as modeled in the computer program UTEXAS2 (Edris and Wright 1987) was used to perform the slope stability analyses. The bonded zone was designed for 30 ft (9.14m) and the unbonded lengths ranged from 27 ft (8.23m) to 83 ft (25.3m). Tendon allowable design loads ranged from 235 kips (1,045kN) to 358 kips (1,592.5kN).

Anchor Design

High capacity grade 270 ksi (1,861MN/m^2), A416 low relaxation 7-wire strand tendons were used. To increase competitiveness, the strand diameter was not indicated in the plans. Only the design load and criteria were specified, which allowed the contractor to determine the number of strands using the strand diameter of his preference. Hardware, jacking equipment, and anchorages were contractor designed to give the contractor the flexibility of using methods and materials which he felt best suited the site. Tendons were sized for the maximum load expected during construction

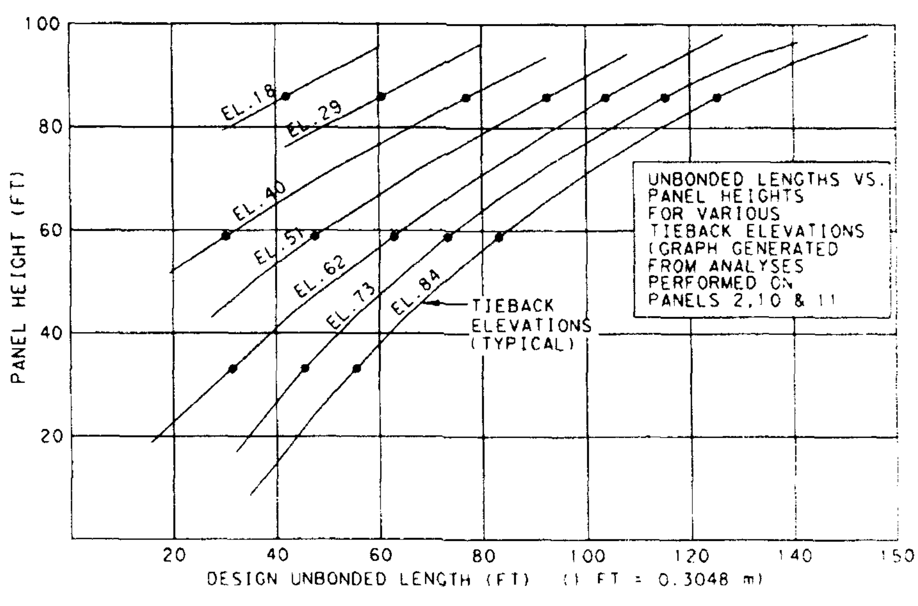


Figure 7. Tendon unbonded lengths

which normally was the test load of 1.5 times the design load.

Instrumentation

The wall was instrumented (Figure 5) to monitor performance during construction and throughout its service life. Instrumentation consisted of embedded inclinometers, strain gauges on the wall reinforcement steel, tiltmeters, horizontal multi-position bore hole extensometers, and tieback anchor load cells. The strain gauges were spaced every 5 ft (1.5 meters) on vertical reinforcing bars located at the inside and outside wall face.

Prediction of Performance

The performance of the temporary tieback wall was predicted using the computer program CBEAMC (Dawkins 1982) which models soil-structure interaction using beam-column structures with one-dimensional linear and nonlinear spring supports, based on the Winkler hypothesis. This analysis method was selected because of the relative ease (compared to two-dimensional finite element methods) with which the beam-column analysis can be made. Input soil parameters are shown in Table 1. Wall behavior was analyzed using a "one-step construction" model that did not take into consideration the construction sequence (Figure 8). Initial loads on the wall were at-rest soil loads with the tendons stressed to the design load. Hand check procedures were used to determine moment and shear loads for the various construction loads.

Terzaghi (1955) showed that deflections are sensitive to the value of L_h because

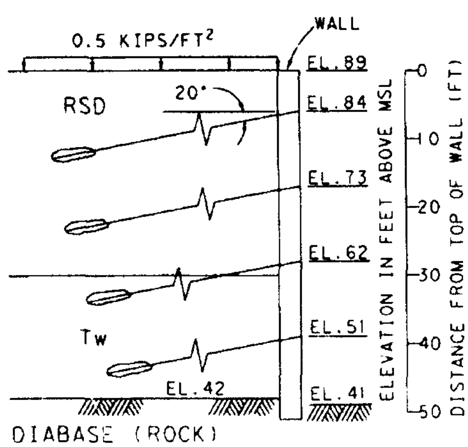
TENDON ELEVATION	BONDED LENGTH (FT)	UNBONDED LENGTH (FT)	DESIGN TENDON LOAD (KIPS/LF)
84	30	74	28.1
73	30	64	28.1
62	30	53	28.1
51	30	37	35.8

STRUCTURAL PROPERTIES

CONCRETE MODULUS OF ELASTICITY = 499,590 KIPS/FT^2

CROSS SECTION MOMENT OF INERTIA = 2.25 FT^4

1 Ft = 0.3048 m
1 KIP = 4.448 kN
1 KIP/FT^2 = 47.88 kN /m^2



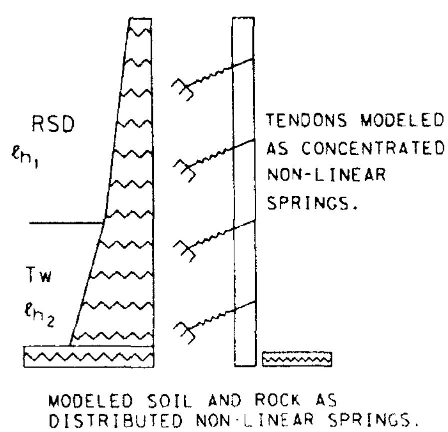


Figure 8. CBEAMC model

displacement is inversely proportional to L_h and that moments were much less sensitive to L_h because moments were proportional to the fourth root of L_h. With this in mind, a parameteric study was done and the results showed, for even the most severe soil condition, the predicted deflection was acceptable (Figure 9).

The stiffness of the soil springs (Figure 10a) used in the CBEAMC analysis is a function of Terzaghi's constant of horizontal subgrade reaction. Terzaghi (1955) called this stiffness the coefficient of horizontal subgrade reaction (k_h) and Haliburton (1971) called this soil parameter the soil modulus (E_s) shown in equation 4.

$$E_s = L_h \left(\frac{X}{D} \right) \tag{4}$$

where L_h = constant of horizontal subgrade reaction, X = equivalent depth of material which will provide the actual effective overburden pressure acting at the elevation of interest, and D = contact area or effective contact dimension (Haliburton 1971). For our case, D was taken as the average vertical distance between the tendons. E_s is the elastic portion of the nonlinear soil spring and is used to establish the displacement (y) required to achieve the active (P_a) and passive (P_p) stress states for the soil spring. Equation 5 shows the relationship for finding the displacement required for the active load condition (Haliburton 1971).

$$y_a = \frac{(P_o - P_a)}{E_s} \tag{5}$$

"Rock springs" (Figure 10b) were used for sections of the wall to be embedded in the diabase. The rock springs were modeled as a 1-ft square (.093m^2) and 2-ft-long (.609m) rock struts that only supplied passive load. The length of the rock strut was set equal to the depth of wall embedment. The maximum allowed load in the rock strut was set at 0.8 of the unconfined compressive strength of the diabase at 16,000 psi (110.3 MPa/m^2) and the modulus of elasticity was 6,040,000 psi (41.641 GN/m^2) per inch (.0254m).

The tendon springs (Figure 10c) were modeled with the length of the tendon equal to the unbonded length plus one-half of the bonded zone. The initial load and elongation in the tendon spring at the start of the analysis was set at the design load with the corresponding elongation of the tendon at that load. Tendon failure was modeled by allowing the tendon load to go to zero after it exceeded 0.8 of the guaranteed ultimate tensile strength (GUTS) of the tendon.

Construction

The 3-ft-thick (.9144m) concrete tieback slurry wall consists of nine 20-ft-long (6.1m) structural panels with embedded reinforcement steel, anchor plates, and instrument pipes. After all panels were completed and cured, excavation in front of the wall began. The excavation did not extend more than 5 ft (1.5 meters) below a given row of soil anchors until they were tested and locked off. A Klemm KR805 Double-head-overburden drilling unit with a down-the-hole hammer with a button bit and "lost crown" was used to drill a 5-1/4 in. (133.35mm) hole with a 5-in. (127mm) casing following immediately behind the bit.

The drill steel and down-the-hole hammer minus the lost crown were removed and tendons installed. Spacers and centralizers were installed on 10-ft centers in the bonded length to keep the strands apart, allowing grout to surround the strands. The contractor elected to pressure grout the anchorages instead of using the specified tremie method. A grout and flush tube was placed parallel with the strands. Grout was pumped into the bonded zone (bottom of the tendon) until it returned out of the top of the hole, then the hole was closed and pressure grouted at 90 psi until grout refused to pump. Next, water was pumped into the flush tube which was located 20 ft above the bonded zone, until water exited the top of the hole. During the performance and proof testing of the 38 installed tendons

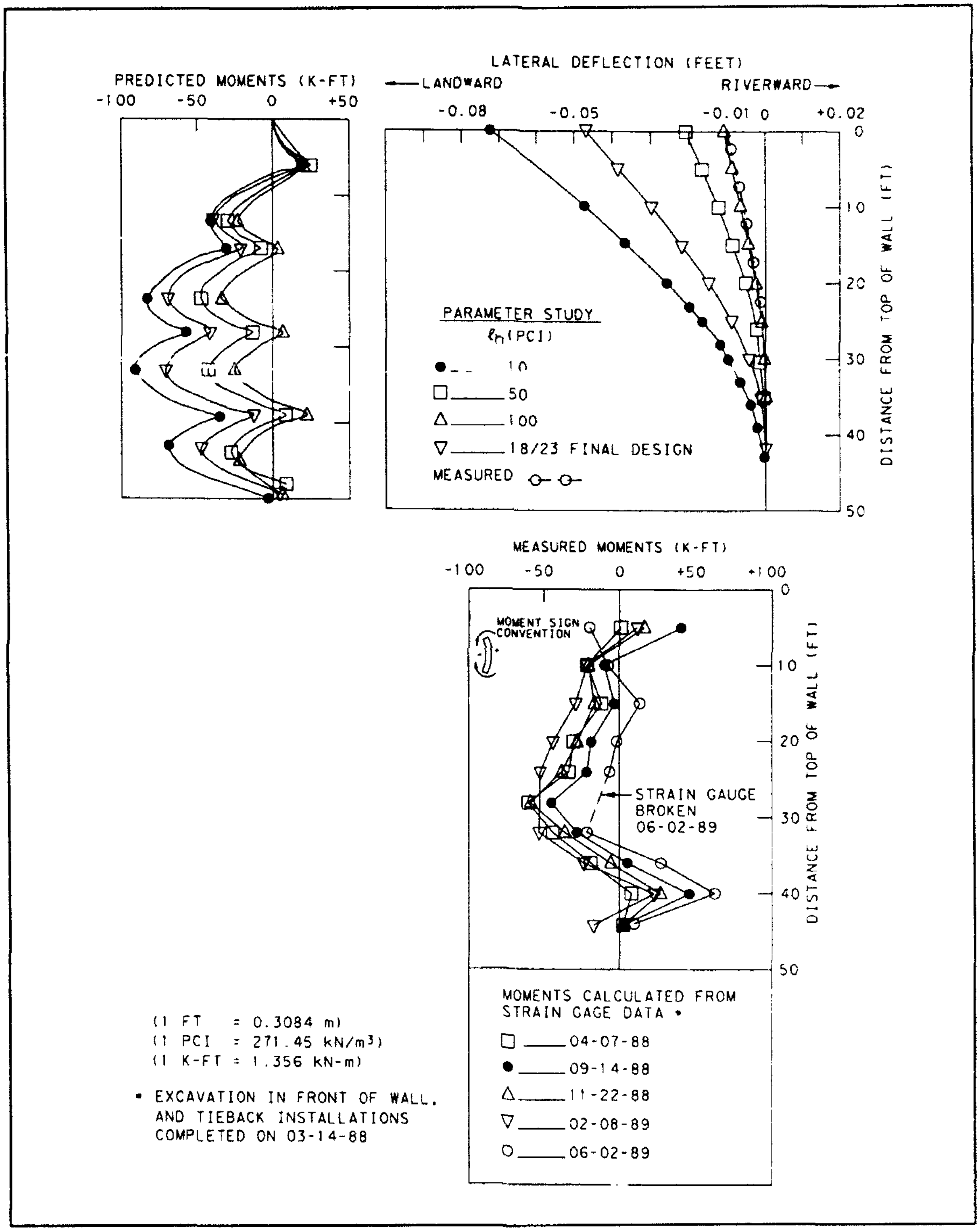


Figure 9. Predicted and measured performance

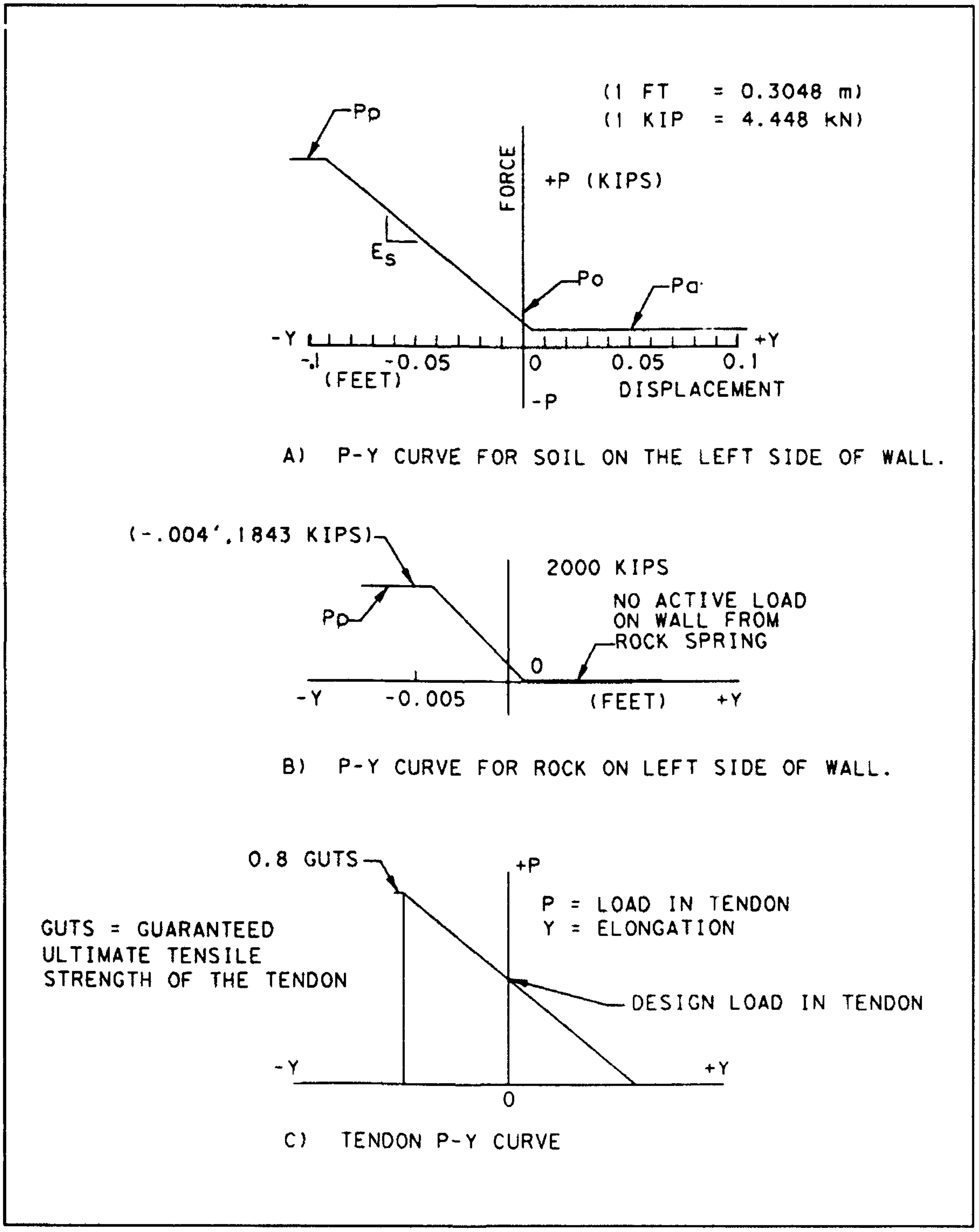


Figure 10. Typical P-Y curves used in the CBEAMC model

there was one pull out failure and two creep failures. All three failed tendons were successfully locked off at a reduced load. After the first tendon failed, the grout to soil interface was increased to 40 ft (12.19m) while the tendon to grout bonded zone remained 30 ft (9.144m) long by leaving the grease filled sheathing on the strands as per the original design.

Measured Performance

Wall deflections were measured with inclinometers. Vibrating wire strain gauges were used to gather data to determine moments. Unfortunately the strain gauge readings taken during the construction of the wall were not taken at the appropriate stages and the data is invalid. The measured deflection at the top of the wall was 0.01 ft (3mm) landward and just slightly less in magnitude than the 0.05 ft (15mm) predicted by the CBEAMC analysis (Figure 9). The moments predicted by the CBEAMC model were approximately equal to the measured moments, but the measured trends did not indicate the reduction in the moments at the supports that were predicted by the CBEAMC analysis (Figure 9). Maximum moments for the final load condition occurred at elevation 62 (18.9m) near the point of maximum bending of the wall. The measured moments have varied with time. This variance appears to be related to changes in ambient temperature and instrument data scatter.

The load cell data (Table 2) show that there was a 3 to 6 percent drop in load from lock off to the initial permanent load in the tendon. The data also show the loss from the lock off load to the long-term load is between 4 to 9 percent of the lock off load. The initial permanent loads were entered into the CBEAMC model and there was no significant difference between deflections and moments from the "measured" tendon loads versus the design loads used in the original prediction.

Table 2
Load Cell Data

Tendon	Lock Off Load kips[1]	Inital Load kips	Long Term Load kips
I-84	272	265	260
I-73	292	275	265
I-62	290	—	270
I-51	356	—	—

[1] 1 kip = 4.446 kN.

Using the data from the upper three load cells to back-calculate soil loads on the wall shows the soil load to be in the range of 2.2 to 2.3 kips/sq ft (105.3 to 110.1 kN/m^2). This compares favorably with the design load of 2.1 to 2.4 ksf (100.5 to 114.9 kN/m^2) and confirms that applied anchor loads, if of sufficient magnitude, can dictate the soil pressures applied to the wall.

Model Calibration

By increasing L_h to 100 pci (27, 145 kN/m^3), the CBEAMC one-step model was able to closely match the measured wall deflections. This was a five fold increase of L_h from the design values and corresponds inversely to the five fold decrease of the measured deflection as compared to the predicted deflection (i.e., $y \propto 1/L_h$). The calculated moments were still in the range of the measured moments (Figure 9). Increasing the friction angle of the soil values used to an average value more in line with a stiffer soil had only a minor effect when compared to increases in L_h (Figure 11). The soil load on the wall calculated by CBEAMC remained basically the same for all values of L_h (Figure 4).

Design of Remaining Walls

Remaining tieback walls at the lock project were designed using L_h values for RSD which were twice those used for the temporary tieback wall. The revised L_h values were 36 pci (9,772.2kN/m^3) for moist RSD and 26 pci (7,057.7kN/m^3) for submerged RSD. These values were based on the comparison of the predicted deflections and moments for panel 6 with the measured deflections and moments. These data indicate that the L_h values might have been tripled or quadrupled

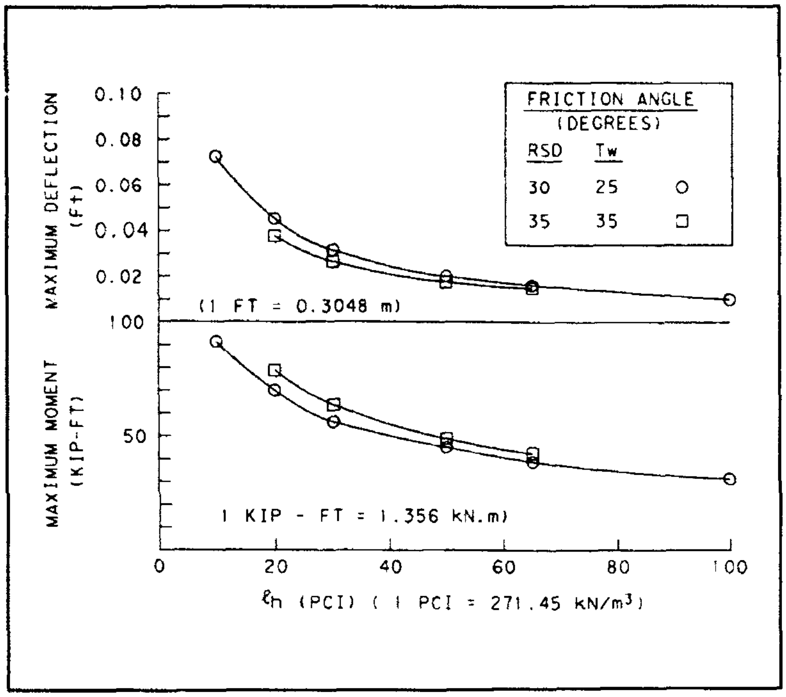


Figure 11. Comparison of friction angle and L_h on deflection and moments

without adversely affecting wall performance. However, due to the importance of the upstream walls, the lack of similar precedents from previous projects, and the likelihood of significant variability in materials at the site, it was decided that increasing the values by more than two times would not be conservative. Both of the L_h values adopted for RSD in design of the additional walls are greater than the highest L_h values provided by Terzaghi (1955). With additional information obtained from the design and construction of future walls in various types of materials, it may become reasonable to suggest the use of even higher L_h values for walls designed using soil-structure interaction methods.

Conclusions

The soil structure interaction beam-column method is a reasonable procedure for analysis of a stiff tieback wall that is designed for at-rest soil loads. An upper bound value of the Terzaghi constant of horizontal subgrade reaction should be used for this type of wall. Use of low bound values for the subgrade constant will overestimate movement and moments. The data reconfirm that deflections will be minimal for a tieback wall designed using the current empirical procedures to derive the at-rest soil loads.

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Mud Mountain Dam Intake Works Replacement

by
Paul C. Noyes, PE[1]

Abstract

The existing intake works were discovered to have inadequate capacity to resist seismic loading in accordance with new criteria. New estimates of debris quantities and sediment deposit also indicated potential failure of the trashrack structures with consequential loss of control in drafting of the pool. A new intake was designed to correct these problems. Incorporated into the new design were improvements to access, increased safety of debris removal methods, and fewer environmental impacts. The various design assumptions, criteria, and analysis procedures are presented herein.

Introduction

Mud Mountain Dam is located on the White River approximately 50 miles southeast of Seattle, WA. The 425 ft high earth core, rockfill dam was constructed from 1939 to 1948 with a 5-year interruption during World War II. The purpose of the dam is flood control. The old outlet system consisted of two separate tunnels each with individual trashrack towers. There is a 9-ft horseshoe shaped tunnel with upstream controls and a 23-ft-diam tunnel which trifurcates at the tunnel mid point into 3 and 8 ft and 6-in.-diam penstocks with Howell-bunger valves at the downstream end. There is no vehicular or personnel access to the trashracks during high water. The inlet for the 9-ft tunnel is at the bottom of the reservoir. The entrance to the 23-ft tunnel is approximately 75 ft above the 9-ft tunnel entrance to avoid being plugged by debris and to avoid the erosion problems of the 9-ft tunnel. Normal debris removal and trashrack clearing is a barge mounted operation with safety concerns.

A concrete cutoff wall was recently installed through the core of the dam and keyed into the rock foundation. The purpose of the cutoff wall was to intercept seepage resulting from differential settlement at the near vertical abutments. A slurry trench technique was used with a continuous type excavator known as the hydrofraise. At 402 ft deep the cutoff wall is recognized as one of the deepest walls of its kind in the country.

Deficiencies of Old Design

Design of the original intake towers was based on the 0.15 g static coefficient method. A seismic study was conducted for the site and new criteria were established. Design earthquake was revised to 0.24 g for the operational basis earthquake (OBE) and 0.35 g for the maximum credible earthquake (MCE). A subsequent reanalysis of the existing towers revealed that they would be significantly overstressed and the trashracks would sustain extensive damage under the new design earthquakes.

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A study was also made of the probable maximum precipitation. Resulting debris from the upgraded spillway design flood would fail the trash rack. Assuming blockage of trashrack area below the sediment level, there was also inadequate trashrack area to effect pool regulation.

New Intake Works Design

A General Design Memorandum was completed in 1986, followed by a Feature Design Memorandum in 1989. Proposal was made that the old structures be replaced. The new single intake tower (Figure 1) will have three inlets at various elevations. Two inlets will connect to the 23-ft tunnel, the other to the 9-ft tunnel. A hydraulic model at 1:30 scale was built and tested at WES. Adequacy of the design was confirmed. The lower inlets to the 23-ft-diam tunnel will not permit accumulations of sediment as in the past, which, when released in concentrations, has detrimental effect on fish. The new centrally located tower will facilitate access and reduce costs of debris removal operations over the life of the project. Working platforms are provided at elevations 935 and 960 from which a mobile crane could work to clear debris from the lower trashracks. The 960 level will remain dry for most of the year. From the 960 elevation a cylindrical trashrack tower extends to elevation 1100, the top being 20 ft above the maximum debris/sediment level. An elevation 1100 pool is approximately a 20-year event. An upper access road and vehicular bridge is provided to the top of the trashrack tower for cleaning the upper portions. Maximum pool is at elevation 1252. Because of anticipated air demand at the regulating gates, air vent shafts were provided. Alternatives to the free standing shaft included sinking a shaft through the bank, and laying the shaft on the bank surface. The excavated shaft was more expensive. And the surface shaft was rejected because of surface instabilities on the bank. Personnel access was provided with all pools via a stair well adjacent to the air shafts. From the base of the intake structure at elevation 895 the trashrack extends 205 ft. And from the roof of the gate chambers at elevation 976, the air and access shafts will cantilever 284 ft. The project has two bridges. A vehicle bridge at elevation 1100 provides equipment access to the top of the trashrack for the purpose of clearing debris. The bridge is approximately 100 ft long, consisting of standard prestressed concrete girders. The second bridge is strictly for pedestrian use. It is at elevation 1260, above high pool and provides emergency access under all conditions. The length of the bridge is approximately 250 ft, consisting of two equal spans. Consideration was given to using a steel truss, single span bridge, but preliminary cost estimates indicated that two spans of 125 ft using standard concrete girders would be cost effective even with the additional cost of a pier. The pier is approximately 100 ft high and will be made up of precast segments post-tensioned together.

Loadings

Various loadings consisted of live load (LL) such as vehicular traffic, dead load (DL), sediment load (SL), debris load (DB), wind load (WL), hydrostatic load (HL), uplift (UF), earthquake loads (EL), and hydrodynamic forces (HF). Factored combinations of loads were in accordance with ETL 1110-2-312 as follows:

$$U = 1.5DL + 1.9LL$$

$$U = 0.9DL + 1.9LL$$ [1]

$$U = 1.5DL + 1.0LL$$ [1]

All loads other than dead loads are considered live loads. Earthquake loads were not combined with any other live loads, nor was wind. Sediment, debris, and hydrostatic loads were combined. Hydrodynamic loads were only applicable to gates. For short duration loads with low probability of occurrence:

$$U = 0.75(1.5DL + 1.9LL)$$

[1] Since DL and LL were found to not act in opposition, these load combinations were not used.

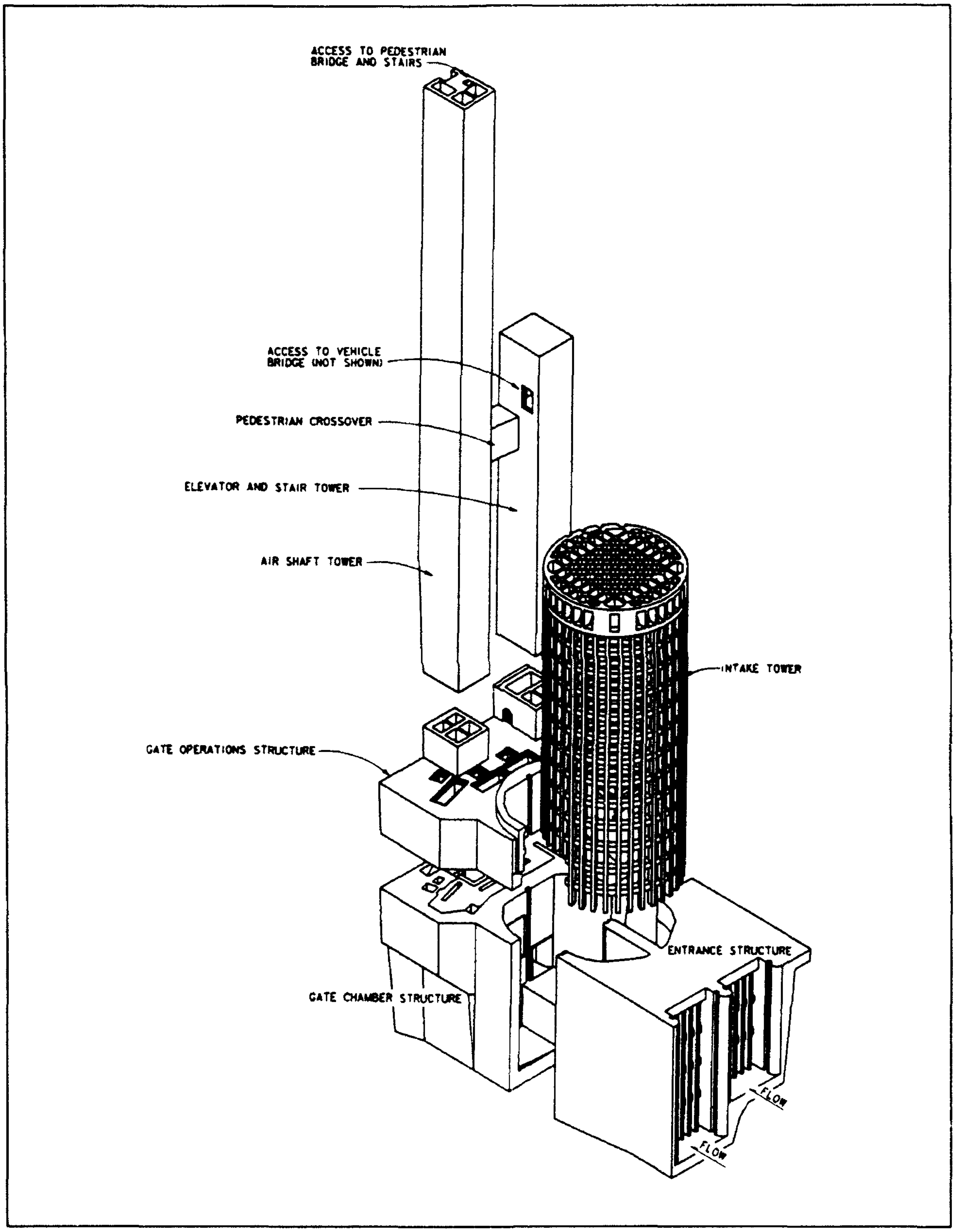

Figure 1. Mud Mountain Dam intake works

$U = 0.75(0.9DL + 1.9LL)$ [1]

$U = 0.75(1.5DL + 1.0LL)$ [1]

For extremely low probability loads such as the maximum credible earthquake (EL2) and sediment load (SL2) resulting from a spillway design flood, factors were reduced even further than those in ETL 1110-2-312. The ETL does not give specific information on how the two levels of earthquake are to be treated, nor does it differentiate between the various degrees of low probability as represented by a project design event versus spillway design event.

$U = 0.75(1.5DL + 1.9HL + 1.6SL2)$

$U = 0.75(0.9DL + 1.9HL + 1.6SL2)$ [1]

$U = 0.75(1.5DL + 1.0(HL + SL2))$ [1]

$U = 0.75(1.5DL + 1.33EL2)$

$U = 0.75(0.9DL + 1.33EL2)$ [1]

$U = 0.75(1.5DL + 1.0EL2)$ [1]

By coincidence the factored OBE of 1.9 (0.24g) and the factored MCE of 1.33(0.35g) were essentially equal; therefore, only one level of earthquake was used. That earthquake being the OBE or EL1 since the structural performance criteria were more stringent under this loading. Uplift pressures were assumed at 100 percent. Sediment elevations were estimated at 1015 for SL1 and elevation 1080 for SL2. Maximum head drop across the trashrack was estimated at 50 ft assuming a blockage percentage of 75 percent. Bridge LL assumed 10 ft deposits at a unit pressure of 224 lb/sq ft. Debris loads governed for the bridge and head drop pressures governed for the trashrack top deck.

Analysis

Separate analyses were done on the individual major components. Analysis of the tower used a substructure approach. The methods of analyses were tailored to the type of structure and the concerns held for each. The massive tower base (below elevation 960 at the entrance and below elevation 976 at the gate chambers) is surrounded on two sides with bed rock, and rock fill on the third side. Only a static analysis will be performed on the structure base since dynamic response is considered insignificant. A finite element analysis was to be performed on the base. A concerted effort was made to maintain a certain simplicity, but with varying degrees of success. To date finite element results have not been obtained. Difficulties with finite element method were primarily systems and software related. The fallback position has been the use of hand methods applied to an idealized, simplified structure and assumed load paths.

The cylindrical trashrack structure is a moment resisting space frame structure consisting of columns also serving as trashrack bars, and a series of internal compression rings supporting the trashrack bars under hydrostatic and debris loading. The rings were set back to increase the tendency for debris to slide to the bottom. This offset created a eccentric column beam joint that made it difficult to obtain the necessary overlapping shear area. A variance from standard practice was needed because of the relatively small columns and large beam connections. Grating is used on the top surface to maximize open area, yet providing a driveable surface for various vehicles including the mobile crane for debris removal operations. Both static and dynamic analyses were conducted on the trashrack structure due to its slenderness and flexibility. Because of its high level of redundance, a ductility factor of 2 was permitted. Consideration was given to use composite sections of either steel encasement or nosing in the lower portions where river gravels and sediment present the greatest erosion potential. Steel nosing was used at the lower elevations.

The air vent and access shafts are cantilever concrete box sections. A crossover box

[1] Since DL and LL were found to not act in opposition, these load combinations were not used.

girder connects the two at mid height. This crossover provides personnel access from the stairs adjacent to the air shafts over to the access shaft that provides primary access from vehicle bridge level at elevation 1100. Watertight doors will be sealed during a flood event. Because of the need to provide a watertight connection at the crossover, a moment connection was necessary. The result was an unsymmetrical "H" shape structure. Initial attempts were made to anchor the shaft to the adjacent rock wall which extends to elevation 1040. Very high reactions resulted at the assumed rock bolt locations. They were high, in fact, that it would have been impossible to provide for anchorage even with the largest prestressed tendons. The first impression was that there was an error in the results. After several more computer runs with differing anchor spacing and spring rates, the source of the extremely high reactions was finally attributed to the effect of lever action with a couple moment arm equal to the distance between rock anchors. Alternate methods considered for bracing the air shaft tower were to tie the tower to the trashrack, to tie it to the bank via the bridge, and a combination of the two. The tie to the trashrack was discarded due to the relatively flexible trashrack and the eccentricity of the connection. The support tie to the reservoir bank was rejected because of the difficulty determining modulii of the soil and the difficulties accommodating thermal expansion in the bridge. The final design selection was an unsupported cantilever with a seismic joint between the structure and the rock wall. By uncoupling the various pieces of the structure, the analysis was simpler, and hopefully the performance of the structure is more predictable.

The trashrack tower was analyzed using Intergraph-RandMicas (IRM). The model consisted of 1216 line elements and 616 nodes. The model of the cylindrical structure was generated taking advantage of symmetry. The dynamic analysis employed Ugss, Lysmes, and Seed's (1974) response spectrum with 5 percent damping. Because of the nonsymmetric base, seismic loads were applied in the x and y as well as an intermediate angle. Vertical acceleration was applied at two-thirds the horizontal magnitude. Under seismic loading a pool to elevation 1070 was assumed with a return frequency of approximately 20 years, a pure judgement call. Effects of the reservoir were accounted for by lumping the effective mass at the nodes.

The air shaft tower and access shaft tower connected by the moment resisting crossover girder were simply modeled by only 84 line elements. To minimize base stresses, the bridge has slip bearings at the crossover. The bridge spans is pinned at the bank abutment. The air shaft base was assumed fixed while recognizing that although the base is massive concrete, it does have some flexibility. But the assumption of fixity is considered conservative because the resulting period of the structure is shorter and closer to the period of peak response.

Attempts were made to auto-generate the model of the tower base using the Finite Element Modeler (FEM) software by InterGraph. A test program was set up to use and evaluate this program written primarily for mechanical components, in the application of massive structures. The effort to date has not been entirely successful for a variety of reasons. In spite of attempts to keep the number of elements at a manageable number, the model became very large. Due to a lack of time and budget, efforts to continue the use of FEM were concluded. The potential remains good for the auto-generation of elements, saving months of labor. The use of such a program on the first attempts should be limited to small, simplified models on projects without tight budgets or critical schedules. The fallback position has been to take a more qualitative approach, relying on judgement in determining steel requirements around voids, simplified assessments of load paths, and unit beam strips of the thin diaphragms.

Value Engineering

The original concept of the operating chamber had a 90-ft high ceiling. The space provided for the installation of bridge crane for

handling the removal of equipment, a somewhat similar arrangement to a typical powerhouse. Prior to final design, a value engineering effort was undertaken. The result was to lower the ceiling/roof by 65 ft and to eliminate the bridge crane. The new scheme called for the utilization of an existing mobile crane with access through roof hatches positioned directly above the gates and cylinders that may need to be removed at some point during the life of the project. Because the remaining wall spans were reduced, the wall thickness was also reduced. Costs were incurred for a new road ramp for the mobile crane. Net savings was estimated at $1.6 million.

Construction

To avoid delays to construction, the work was divided into two phases. The first phase consisted of an access road, initial intake tower site excavation at the higher elevations, cofferdam, and river diversion with temporary trashrack. By constructing in two phases, it permitted time for in-house final design and contract document preparation. By having one contract for site work and the other for structural work, each contract could be awarded to contractors with the particular specialties involved. Given historical precipitation and snow melt, the construction season is March through October. For this 8-month period, the cofferdam was designed to provide protection up to the 10-year event. To construct a cofferdam for year round protection, or even for an additional month of construction time, the costs were prohibitive. Other considerations included water quality and the downstream migration of fish. In-river work was kept to a minimum. Phase I construction is scheduled to take 2 years, and Phase II work is scheduled to take 3 years.

Conclusions

Additional guidance is needed in the area of applicable load factors for both the OBE and the MCE.

ETL 1110-2-312 allows for reduced factors with short duration loads with low probability of occurrence. But it is not clear what to do for short duration loads with high probability of occurrence, for example. Clarification is needed in this area.

There has been much discussion recently of a Puget Sound subduction zone earthquake. Its an earthquake with a broad response spectrum. Peak response is relatively low, but at the period of a typical tower, accelerations are significantly higher than for fault zone earthquake. If adopted, the subduction zone earthquake would make present designs inadequate. If this earthquake presents a credible risk, it must be officially adopted.

Additional guidance is need for long duration earthquakes. The effects of many cycles may have a significant negative effect even with increased damping factored in.

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Analysis of the Seepage Cutoff Wall at Mud Mountain Dam

by
Wayne Kutch, PE[1]

Abstract

This study describes an analysis of the structural behavior of the diaphragm seepage cutoff wall installed at Mud Mountain Dam, Washington. The investigation is presented in the form of two phases, the first of which serves to establish load-deformation characteristics of the dam embankment. For this purpose, a plane-strain finite element model of the dam cross section taken perpendicular to its axis is utilized. Upper and lower modular values for the dam core and rock shell are estimated using equations developed for hyperbolic modeling of soil stress-strain behavior. Loads corresponding to a hydrostatic pressure distribution were applied to the model, and the resulting load-deformation relationships determined. The second phase of the investigation utilizes these relationships as support springs for a planar grid-structure model of the wall itself. Additional spring constants describing the translational and rotational stiffness of the rock abutments were also determined by finite element techniques and then assigned to their corresponding grid perimeter nodes. A hydrostatic pressure distribution was then applied to this model, and the resulting moments and shear forces thus obtained compared with allowable values.

Background

This report describes a deformation analysis of the seepage cutoff wall recently installed at the Mud Mountain Dam Project. Mud Mountain, located on the White River near Enumclaw, Washington, is a rock-fill dam with a rolled earth core. It has a maximum embankment height of 425 ft and a crest length of 700 ft. Construction of the dam began in 1939 and was largely complete when interrupted due to the war effort in 1941. It did not become fully operational until 1949.

The cutoff wall installation was undertaken in response to concern regarding the integrity of the dam's core, prompted by indications of excessive seepage through the embankment. It consists of a horizontal line of 67 individually placed concrete sections, each of which extends the full height of the embankment from foundation rock to the dam crest. Panel widths range up to a maximum of 23 ft with a maximum thickness of 40 in. The panels extend into the foundation rock a minimum distance of 15 ft for seepage control.

The dam embankment was modeled using a two-dimensional plane-strain finite element model of the dam, oriented perpendicular to the axis of the dam, consisting of 146 nodes and 133 elements (Figure 1). Properties for

[1] Structures Design, US Army Engineer District, Seattle; Seattle, WA.

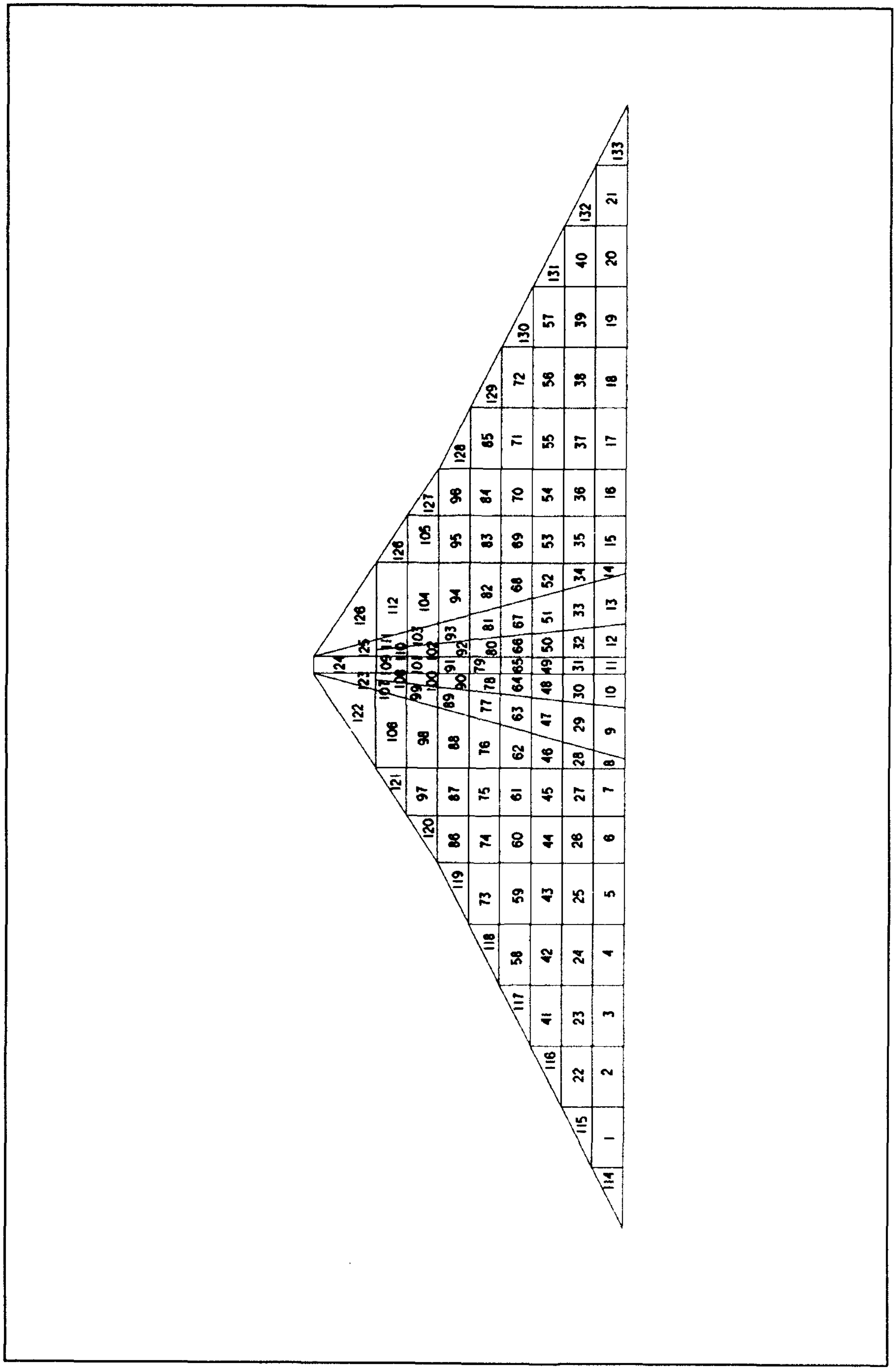

Figure 1. Finite element model of dam embankment

both the core and rock shell material were developed, and hydrostatic forces were applied directly to the element nodes. The model was then analyzed by means of the structural analysis program GTSTRUDL (Georgia Institute of Technology 1981). Load-deformation relationships at the wall location were developed from the computed nodal displacements and utilized for support springs in a planar grid structure model. The grid was analyzed using the structural analysis program CGRID (Dawkins 1987) to determine moments, shears, and deformations in the concrete cutoff wall.

Finite Element Model

Determination of soil moduli

Modulus values for the finite element model were determined using the hyperbolic model (Duncan, Kulhawy, and Seed 1969; Duncan and Chang 1970; Duncan et al. 1980), which relates stress and strain in soils by the equation:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\dfrac{1}{E_i} + \dfrac{\varepsilon}{(\sigma_1 - \sigma_3)_{ult}}} \tag{1}$$

where $(\sigma_1 - \sigma_3)$ is the deviator stress, which is the difference between principal stress and confining pressure, and E_i is the initial tangent modulus of the soil at a particular confining pressure, σ_3. $(\sigma_1 - \sigma_3)_{ult}$ is called the asymptotic stress difference and can be determined by the equation

$$(\sigma_1 - \sigma_3)_{ult} = \frac{(\sigma_1 - \sigma_3)_f}{R_f} \tag{2}$$

where R_f is a dimensionless coefficient called the failure ratio, and

$$(\sigma_1 - \sigma_3)_f = \frac{2c\cos\phi + 2\sigma_3\sin\phi}{1 - \sin\phi} \tag{3}$$

in which c and ϕ are the cohesion intercept and the friction angle for the soil, respectively.

The initial tangent modulus can be determined by the equation:

$$E_i = K\, p_a\, \sigma_3^n\, p_a^{-n} \tag{4}$$

where K and n are dimensionless parameters termed the modulus number and modulus exponent, respectively, and p_a is atmospheric pressure in the same units as E_i.

Attempts at standardizing this approach have lead to the evaluation of a wide variety of soils for the parameters K, N, c, ϕ, and R_f, to be used in the preceding equations (Duncan, Kulhawy, and Seed 1969; Duncan and Chang 1970; Duncan et al. 1980). It was therefore possible, for the purpose of this study, to select representative parameters for the core and shell from the tabulated results of such testing, based on similarities (gradation, unit weight, etc.) to the actual embankment materials.

Embankment parameters

In order to determine the relative contributions of both core and shell to the model behavior, and determine the sensitivity of the model to these parameters, wall behavior was investigated using soil parameters corresponding to two types of core materials and shell materials. Core material parameters investigated correspond to samples taken from the Round Butte Dam core and the Binga Dam core, and were both selected based on their physical similarity with the Mud Mountain Dam core material. The hyperbolic stress-strain curves for both dam core materials are compared in Figure 2. The confining pressure, σ_3, of 10 ksf which was used to develop the plot corresponds to a depth of approximately 150 ft in the embankment. The Round Butte Dam core material is considerably stiffer than the Binga Dam core material, as indicated in Figure 2 by the steeper curve.

The two sets of parameters selected to represent the behavior of the Mud Mountain shell correspond to a diorite rock fill and a conglomerate rock fill. Although the material

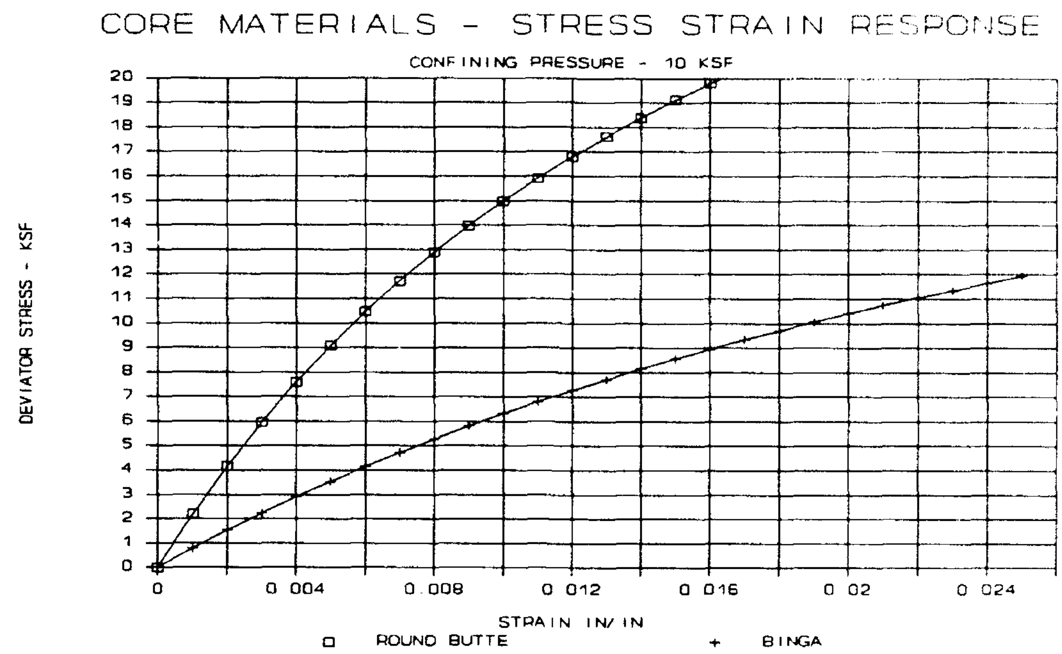

Figure 2. Core σ versus ε at 10 ksf confining pressure

used to develop these parameters consisted of a finer gradation than the large rock in the Mud Mountain shell, both are comparable in terms of void ratio and internal friction angle. A graph of the hyperbolic stress-strain curves for these materials is presented in Figure 3, from which it may be seen that the conglomerate rock fill is stiffer than the diorite rock fill.

Figure 4 shows a comparison between a hyperbolic stress-strain curve and initial tangent modulus, in this case for the diorite rock-fill shell material at an assumed confining pressure of 10 ksf. As can be seen in this figure, the linear function and the hyperbolic function are approximately coincident for very small strains. This similarity was utilized in the effort reported herein, in order to allow the use of linear finite element behavior. Since the two functions diverge with increasing strain, it was necessary to assess the significance of such divergence on the results obtained. The method and results of this assessment will be addressed elsewhere in this report.

Graphs of the initial tangent modulus as a function of confining pressure for both core materials and shell materials are shown in Figures 5 and 6.

Sign convention

The sign convention used for this exercise is positive for downstream displacements, and positive for (passive) soil pressure acting to resist downstream displacements.

Soil behavior modeling

According to the designated sign convention, displacement of the cutoff wall will be in a positive sense when hydrostatic forces are applied. Prior to wall displacement, the at-rest soil pressure will be acting on the downstream face of the wall in a positive sense. In response to positive wall displacement, this pressure will increase as passive resistance is mobilized in the soil. The rate at which passive resistance is developed is expressed by the soil modulus, which, as already indicated, is approximately linear for small strains. In the finite element model used in this analysis, the at-rest pressure coefficient, K_0, of 0.5 is reproduced by assigning a Poisson's ratio of

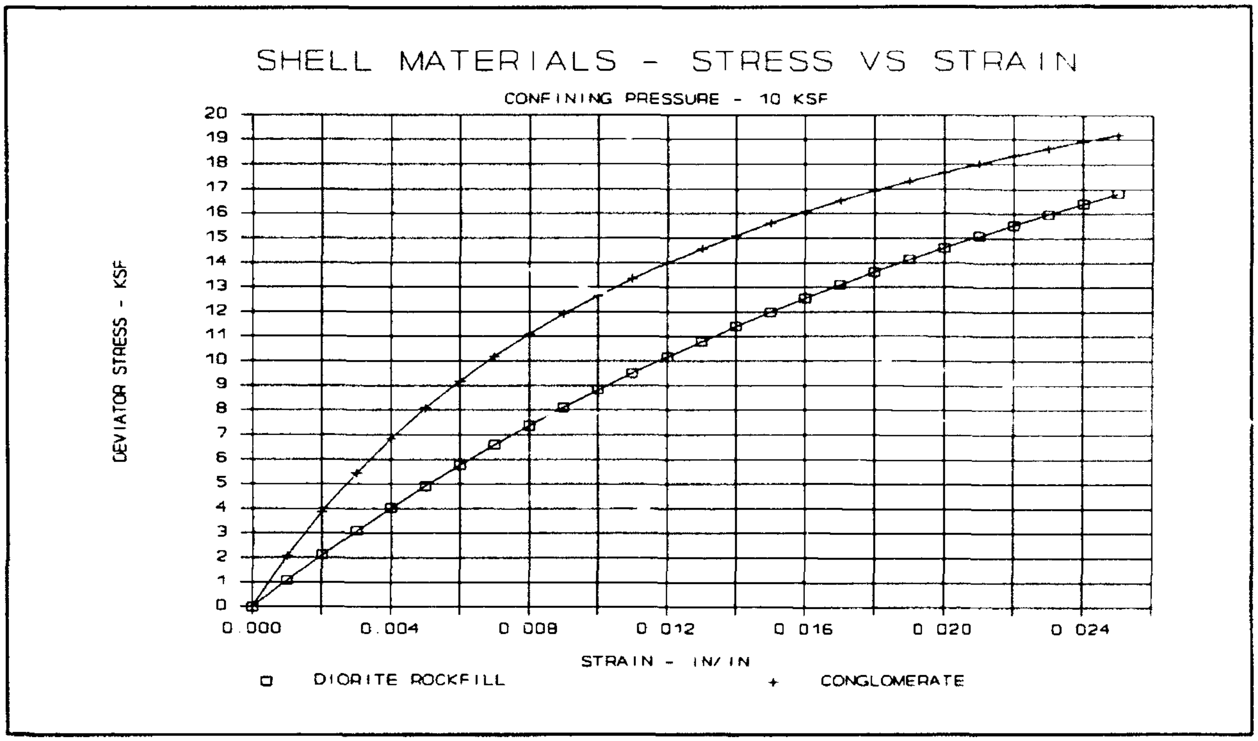

Figure 3. Shell σ versus ε at 10 ksf confining pressure

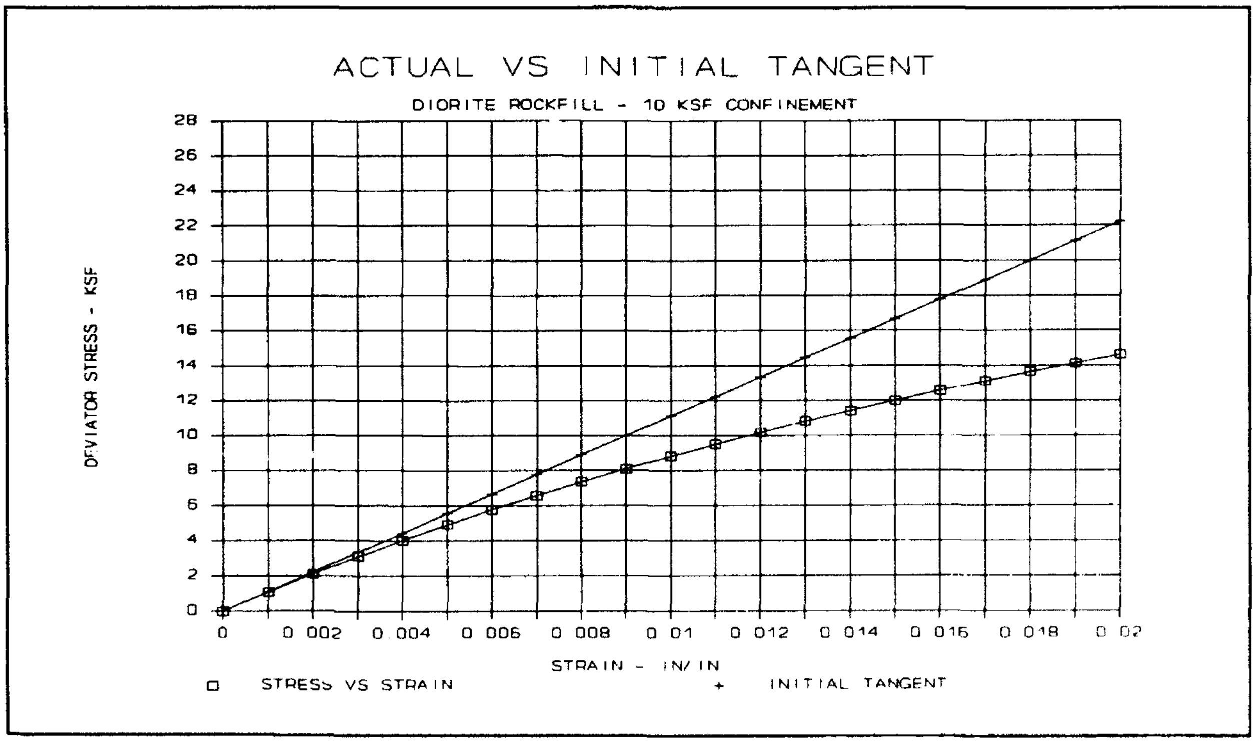

Figure 4. Diorite shell σ versus ε and initial tangent

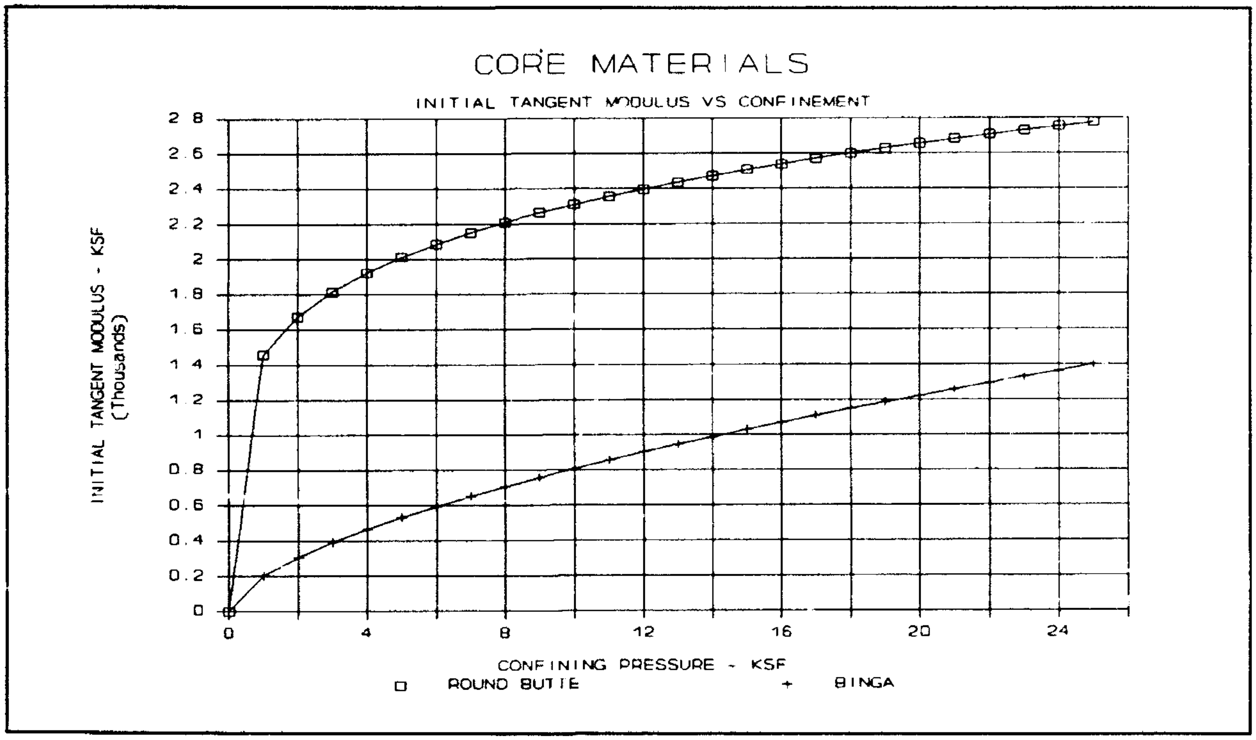

Figure 5. Core materials, E_i versus confining pressure

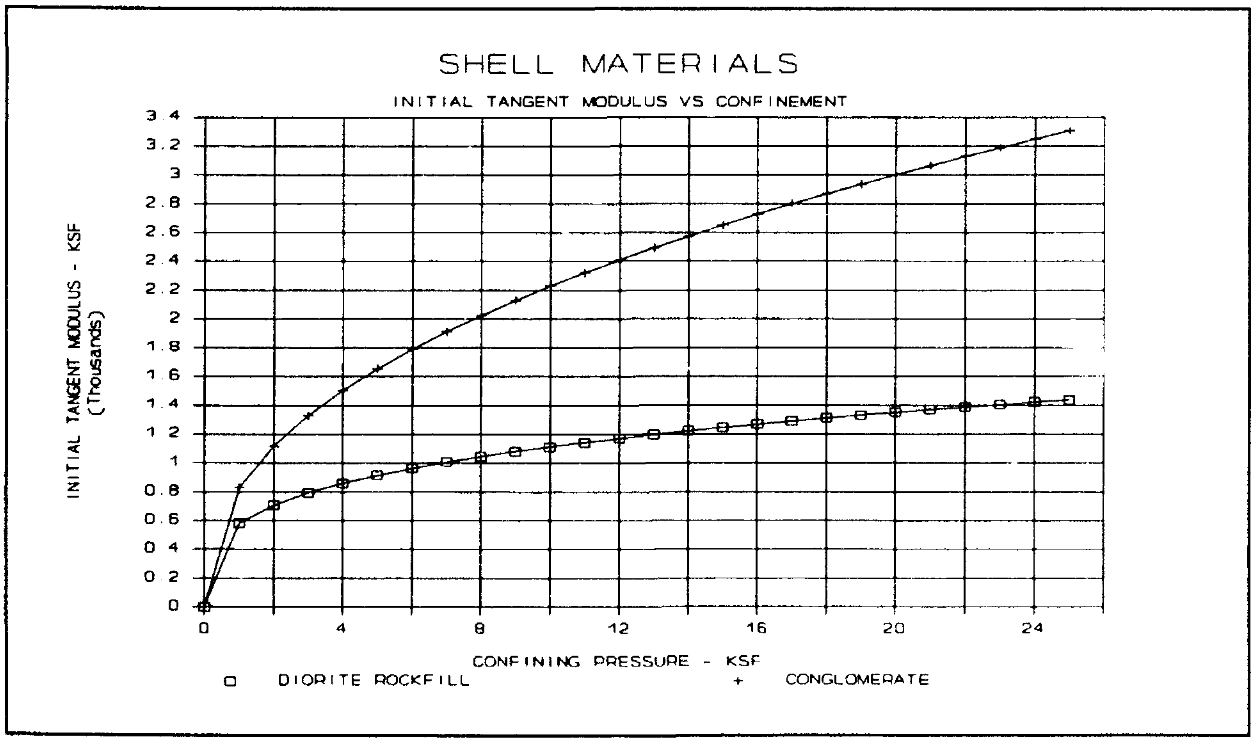

Figure 6. Shell materials, E_i versus confining pressure

0.33 to the elements. Resistance of the downstream soil elements to lateral deformation in the model is slightly higher than the soil modulus would suggest. This occurs as a result of the conditions of strain compatibility in the two-dimensional plane-strain model. The magnitude of the increase in the effective soil modulus produced by this condition is approximately 11 percent, which does not constitute a significant variation relative to the degree to which soil parameters are known.

On the upstream face of the cutoff, soil pressure will act in a negative sense and changes from at-rest to active as positive displacement occurs. The corresponding soil elements in the finite element model exhibit positive strain (elongation) under these conditions. A corresponding decrease in horizontal stress takes place in the upstream elements, which are initially in compression due to Poisson's effect. In the case of actual soil behavior, lateral soil pressure may decrease only until the full active limit state is achieved, at which point additional displacement yields a constant lateral pressure. However, in the linear finite element program used in this analysis, this limitation could not be accounted for in the input stages. It was therefore necessary to verify that the calculated horizontal stresses were within the acceptable range for active soil pressure. The magnitude of full active pressure is expressed by the product of the active pressure coefficient, K_a, and the vertical pressure, and is a function of the embankment geometry and material characteristics. For the sloping dam embankment geometry, the active pressure coefficient is expressed by the equation:

$$K_a = \left[\frac{\cos\phi}{1 + \sqrt{\sin\phi\,(\sin\phi + \cos\phi\tan\delta)}} \right]^2 \qquad (5)$$

where δ is the angle of the embankment measured from the horizontal and ϕ is the internal angle of friction of the embankment material. In the case of Mud Mountain Dam, active behavior is thought to be primarily governed by the rock shell, for which the average internal friction angle is estimated to be approximately 45 deg. Using Equation 5, the value of K_a was determined to be between 0.13 and 0.14, depending on elevation. The requirement that the computed horizontal stress at least equal the product of this factor and the indicated vertical stress at any point provided the verification of the validity of the program results in positive strain regions. As in the case of passive resistance, soil behavior was assumed linear, and no adjustment to soil modulus values was made to account for strain compatibility effects.

Loading condition

The loading condition assumed in this analysis was a hydrostatic pressure distribution corresponding to a pool elevation (el) of 1,150, which approximates the highest pool of record at the project. Loads were applied as concentrated forces at the element nodes in both the finite element and grid structure models.

Finite element results

The results of the finite element analysis indicate that deformations are controlled primarily by the rock shell parameters. Maximum calculated deflection using the conglomerate rock-fill shell was 12.1 in. at nodes 75 and 92, using either core material. Maximum displacement occurred at depths of 280 and 240 ft, or approximately two-thirds the overall depth of the embankment. Differences between nodal displacements using Round Butte and Binga core parameters were generally less than 0.1 in.

For the case of the diorite rock-fill shell, the maximum nodal displacement increased to approximately 19.0 in., also in the 280- to 240-ft-depth range. Once again, the difference in displacement between the two core materials was found to be insignificant. The surprisingly small influence of the core material parameters in the model behavior is thought to be caused by the relatively short horizontal soil column this material provides compared with the large rock shell cross section. A comparison of the deformed shapes of the cutoff wall utilizing the two different rock-fill shell parameters is shown in Figure 7.

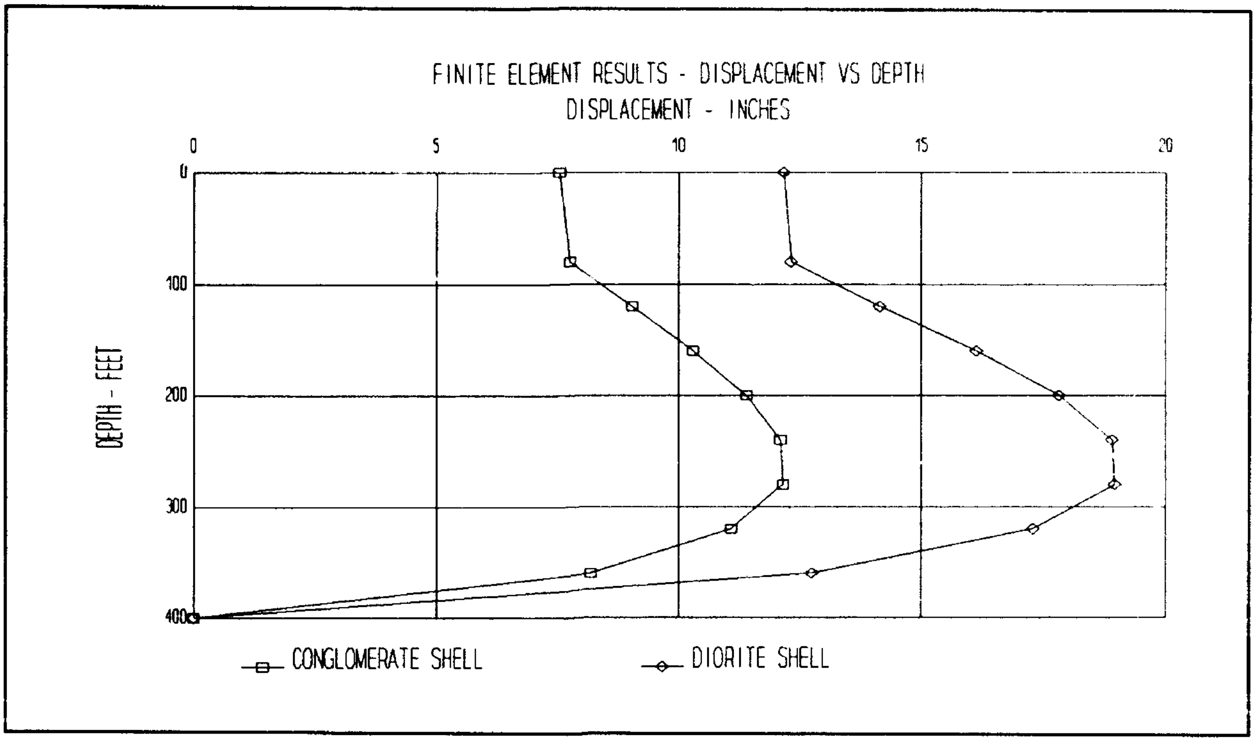

Figure 7. Finite element model nodal displacements

Verification of finite element results

As previously indicated, verification of the finite element results was necessary, particularly in areas of relatively high strains. These areas are primarily located in the dam core adjacent to the cutoff wall. For the case of negative strains, indicating mobilization of passive resistance, verification required determination of the degree of divergence of the theoretical hyperbolic model from the initial tangent modulus. A maximum negative strain of 0.00139 occurs in the core at element 31 using the lower modulus diorite shell parameters. This corresponds to a deviator stress of 3.347 ksf according to the initial tangent modulus of 2,408 ksf used by the program for this element, compared to a theoretical value of 3.146 ksf given by Equation 1, a divergence of approximately 6.4 percent. Since the average negative strain throughout the model is considerably less than occurs at element 31, the overall contribution of such divergence does not appear to constitute a significant source of error for the effort reported herein.

For the case of positive strains, valid results required that lateral pressure not fall below the minimum value for active behavior as given by Equation 5. This requirement was met at all locations, with the exception of several nodes at the base of the transition elements, 123 and 124 (Figure 1). These exceptions are not significant, since they occur in an area of very low stress and are above pool level.

Grid Structure Analysis

General

Force and deformation response of the concrete cutoff wall was evaluated using a planar grid model of the wall-soil system. Embankment characteristics determined from the finite element model were incorporated into the grid as interior support springs. Support conditions at the grid perimeter were also developed to approximate the constraint along the cutoff wall interface with both rock and overburden. Moment continuity was assumed throughout the model for the initial analysis,

corresponding to an uncracked wall. Additional analysis was also performed considering the effects of flexural overstressing at the vertical cold joints between adjacent panels. The loss of flexural resistance resulting from cracking at these locations was simulated in the analysis by the release of bending force about the vertical axis.

Grid configuration

The geometry of the rock canyon at the cutoff wall was approximated by the grid model shown in Figure 8, which consists of 100 nodes and 159 elements. Node spacing was reduced at greater depths, as well as along the rock-wall interface, since these areas are typically associated with higher stress levels, and therefore warrant a higher level of detail.

Element properties

Element dimensions were determined by tributary area and provided as input for property computation by the program CGRID. The modulus of elasticity of the concrete was determined from the ACI code equation $57000\sqrt{f_c'}$, which produces a value of 3,122 ksi for the 3,000 psi minimum compressive strength required by the contract specifications.

Interior support springs

Spring constants at interior nodes representing the deformational resistance of the dam embankment were determined from the finite element results. The concentrated loads used in the finite element model were converted to unit loads according to tributary areas, and then compared with the resulting deformations. The results of this comparison for both the diorite and conglomerate rock-fill shells were then plotted as a function of depth in the embankment, as shown in Figure 9. The units for soil stiffness used in Figure 9 are kips per square foot per foot displacement, or kips per cubic foot, which is referred to as the soil constant, k. Typical values of k for flexible retaining structures in sand range between 5 and 40 kips per cubic foot, with 16 kips per cubic foot for a medium sand (Haliburton 1979). As shown by Figure 9, the

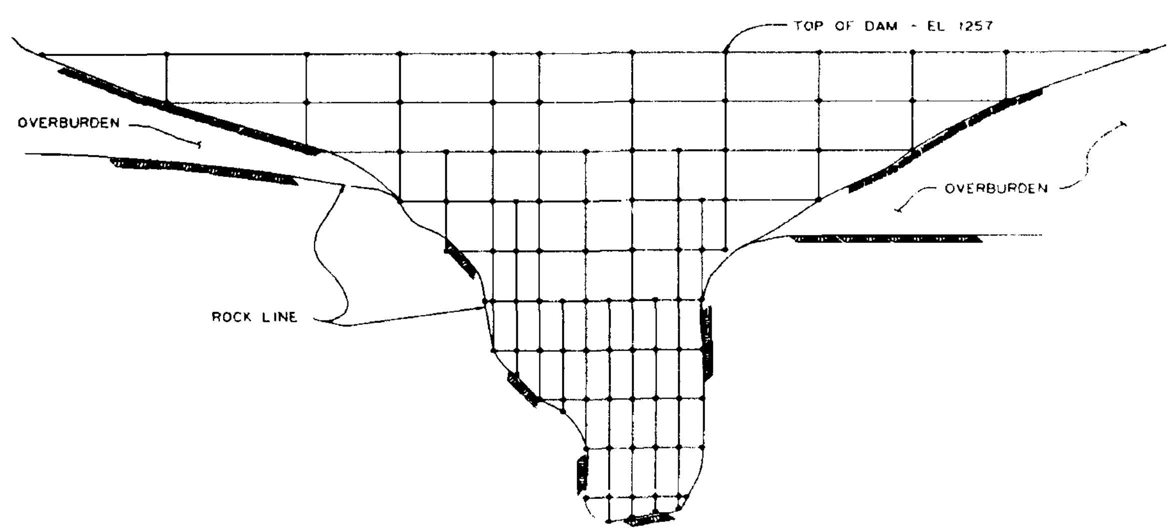

Figure 8. Cross section at axis of dam showing canyon geometry with grid structure model superimposed

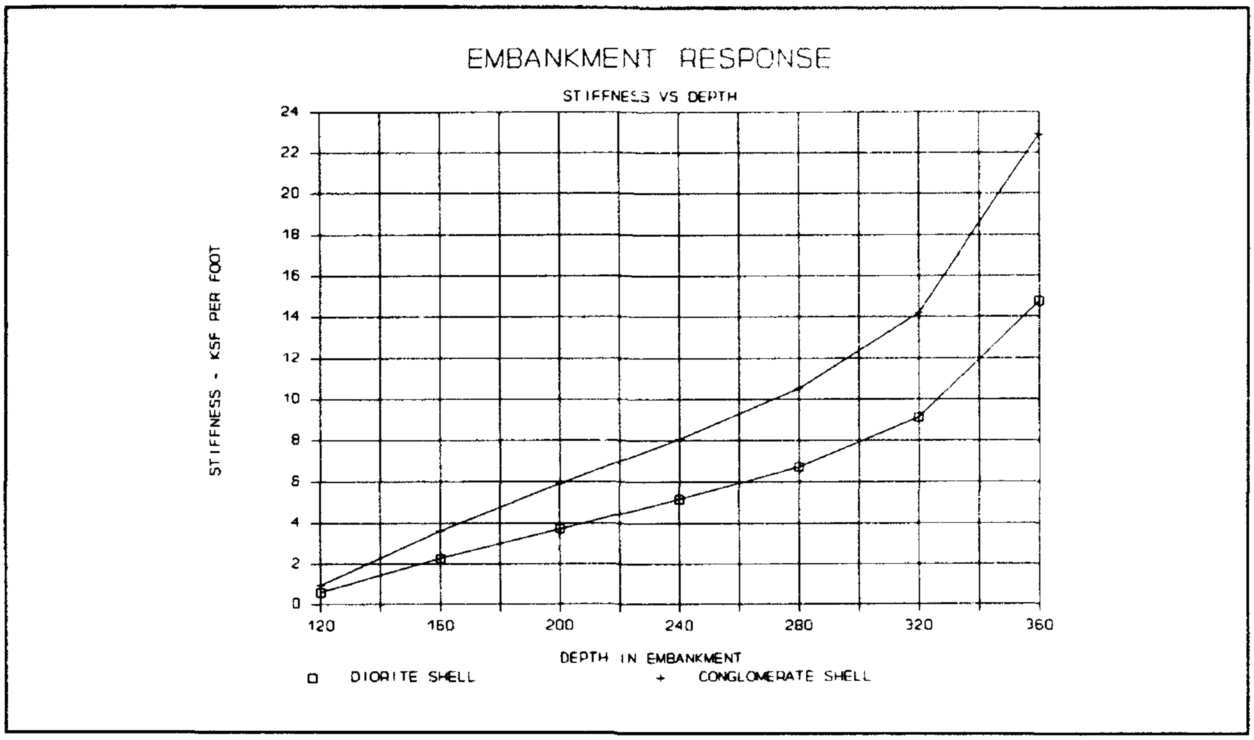

Figure 9. Embankment stiffness versus depth based on finite element analysis results

soil stiffness values obtained by the method described herein compare favorably with these values. Figure 9 indicates a nearly linear relationship between depth and soil stiffness, varying only at the elevation extremes, where perimeter effects would normally be expected. Support spring constants were calculated at each node by multiplying the appropriate tributory area by the spring constant corresponding to the node elevation.

Perimeter model

Mud Mountain Dam is constructed across a narrow canyon with steep rock walls more than 200 ft high on both sides. Perimeter effects under such conditions are a significant factor in the behavior of the wall-soil system. Maximum stresses in the wall will tend to occur due to the relatively high degree of fixity provided by the rock interface. These stresses will be strongly influenced by the ability of the rock to support rotation and translation, and increase as a function of the rock stiffness. Geologic exploration indicates that the rock quality is highly variable throughout the interface area. The modulus of elasticity is estimated to range between 200,000 and 2,000,000 psi, with a median value of approximately 500,000 psi. The results of two plate bearing tests which were performed on the rock abutments (US Army Engineer District, Seattle 1946) indicate a deformational resistance of up to approximately 400 kips per cubic inch.

The behavior of the rock interface is accounted for in the grid structure model as a system of rotational and translational support springs at the perimeter nodes. Spring constants were developed by means of a beam on elastic foundation model of the wall-rock interface, analyzed using the computer program CBEAMC (Dawkins 1982). The model consists of a 1-ft section of wall supported by a distributed linear spring for a distance of 15 ft, which corresponds to the minimum depth of key-in to rock required by the contract documents. Recognizing that it would be impractical to attempt to model the variations in rock stiffness present at the site, a singular spring constant of 4,800 kips per inch per inch was used, which corresponds to the upper limit of the values of rock stiffness determined in the

plate bearing tests. This value was selected to produce higher, hence more conservative, spring constants. Rotational and translational stiffnesses were determined by the application of a moment and force to one end of the model in separate load cases, and relating the displacements thus obtained to the applied loads.

The same beam on elastic foundation model was also utilized to determine perimeter stiffness in the overburden above the rock. The overburden in the project area is a relatively stiff material and a correspondingly high modulus value of 0.417 kips per inch per inch was used as a distributed spring constant in the model. This equates to a soil k value of 60 kips per cubic foot, or 0.035 kips per cubic inch.

Allowable concrete stresses

Flexural and shear stresses were determined from the computed forces, based on section properties for individual grid members according to tributory area. These values were compared with allowable stress levels for flexure and shear. Allowable flexural tensile stress was assumed to be equal to $7.5\sqrt{f_c'}$, according to the ACI Code equation for the modulus of rupture. Allowable shear stress was assumed to be equal to $2\sqrt{f_c'}$, also according to the ACI Code.

Initial results—joint continuity throughout

The initial analysis was run assuming vertical and horizontal moment continuity throughout the structure. The results of this analysis indicate a high incidence of overstress, particularly along the rock interface, and generally increasing in magnitude with depth. Flexural overstressing was found to be more prevalent than shear overstressing, with virtually all members below pool level exceeding allowable. Stresses using the diorite shell parameters average approximately 30 percent greater than those using the stiffer conglomerate shell parameters. Maximum flexural stresses were found to be exceptionally high in some members, ranging to over 41 times allowable for the conglomerate shell, and 50 times allowable for the diorite shell. The magnitude of overstressing evident in this summary indicates flexural cracking of the wall to be inevitable for the assumed conditions of load and support. The maximum shear stress was 13.2 times allowable for the conglomerate and 16.4 times allowable for the diorite. Flexural stresses in horizontal members were found to be influenced not only by depth but also by span, increasing significantly in narrower portions of the model. Such trends were not as easily identified in the case of vertical grid members, which were generally not as highly stressed. However, a notable concentration of higher stress levels was indicated in some of the vertical members in the upper center portion of the model. These members, located at or near the top of the input hydrostatic loads, are subject to only relatively small displacements. Bending stresses are therefore induced in these members as a result of the larger displacements of adjacent members at greater depths. This effect was confirmed by comparison of the deflected shape along the center line of the grid structure model with that of the finite element model, as shown in Figures 10 and 11. This comparison reveals good correlation between the two models for nodal displacements below pool level, with much smaller displacements occurring above pool level in the grid structure model. This deflection geometry is consistent with the additional influence of the wall elements in the grid model in resisting deformation.

Results with moments released

Moment resistance at the vertical panel joints is expected to be influenced by a combination of two factors, joint width and bond strength. A reduced effective section to resist bending will occur due to deviation from exact alignment between adjacent panels, resulting in a reduced joint width. Such deviation is generally acknowledged to be unavoidable with this construction process, although it may be held to a minimum with careful execution of the excavation procedures. Maintenance of panel alignment is enforced by means of a requirement for a minimum joint

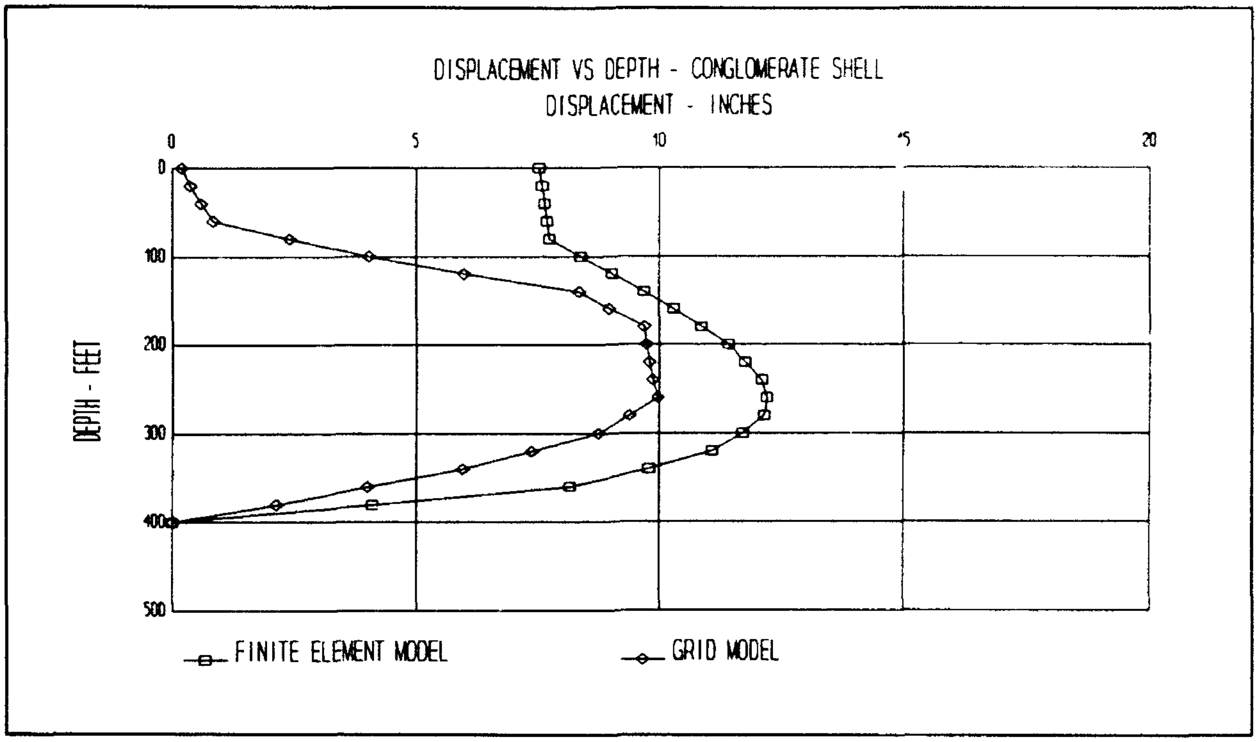

Figure 10. Conglomerate shell, comparison of deflected shape of grid to finite element model nodal displacement

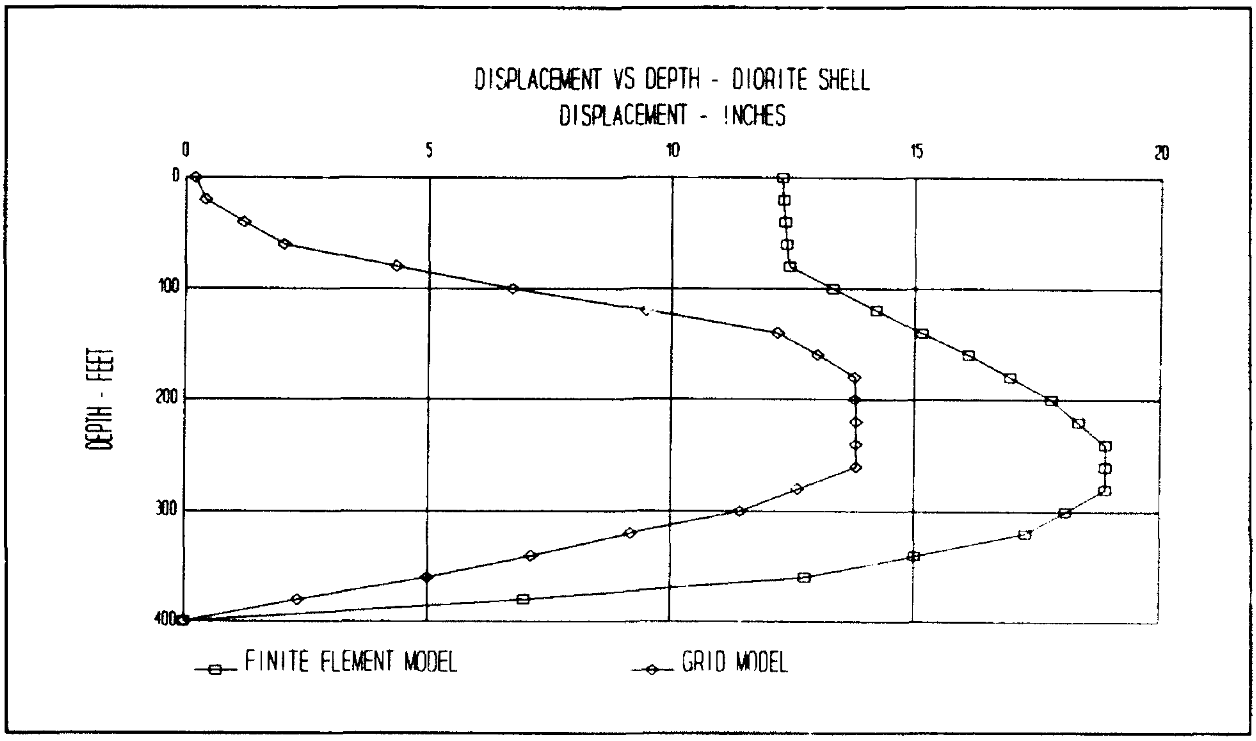

Figure 11. Diorite shell, comparison of deflected shape of grid to finite element model nodal displacement

width in the contract specifications. This requirement was 24 in. in the case of the Mud Mountain installation, representing a 40-percent reduction in effective section depth at the joints, which corresponds to a 64-percent reduction in flexural strength.

Moment resistance at the joint is also affected by the tensile capacity of the concrete bond across the joint face. The texture of the excavated concrete surface produced by the panel installation process is expected to be conducive to the development of good bond strength. However, bond strength at the joint face may be detrimentally affected by inclusions of the bentonite slurry mixture, which can potentially be trapped by surface irregularities along the joint face. The net effect of these two factors on bond tensile strength is unknown, although it is unlikely that such bond will be equal in tensile capacity to continuous concrete. When this factor is considered with regard to the already significant strength reduction possible with a reduced effective section, it can be seen that cracking may occur at the joints under much lower force levels than would be required for continuous concrete.

In order to account for the presence of the panel joints in the wall, model behavior was reevaluated after moment resistance about the vertical axis was released at the joints. Although increases in maximum displacement were generally on the order of 10 to 20 percent at various elevations due to this modification, the most pronounced effect was on the overall deformed shape of the model. By eliminating the deformational resistance provided by the wall stiffness spanning horizontally across the canyon, the model is allowed to assume a deflected shape more closely reflecting an equilibrium position between the hydrostatic driving forces and soil resistive forces. Maximum relative increases in displacement therefore occur close to the wall's perimeter for this case, where the flexural stiffness of the wall previously had the maximum effect in limiting movement. This effect is visible in Figures 12 and 13, which compare plan sections of the deflected position of the wall before and after moments are released. Bending forces in vertical grid members were found to be generally higher than in the fixed end cases, although exceptions can be found. Shear was also found to exceed allowable values, but typically not to the extent of flexural overstress. Horizontal grid elements do not carry either shear or bending moment for this case, as would be expected with the given releases.

Summary

A possible method for the analysis of diaphragm cutoff walls in embankment dams has been described and demonstrated. However, the application of this procedure in the subject investigation required the use of a number of assumptions relating to embankment response characteristics and loading. Although due consideration has been given to the development of reasonable values with regard to these parameters, the applicable site data were insufficient to preclude the use of such assumptions. It is therefore necessary in interpreting the quantitative data thus obtained to be cognizant of the collective degree of uncertainty inherent under these circumstances.

By far the highest degree of uncertainty with regard to this investigation pertains to the assumed loading condition. It is unlikely that the seepage cutoff wall at Mud Mountain Dam will ever be subjected to the full hydrostatic pressure differential of nearly 300 ft used in this analysis, since this would require 100-percent efficiency of the diaphragm wall as well as complete cessation of water migration through the perimeter rock. In the event of the occurrence of such a load, the findings indicate that it would probably not be present for any extended period of time, due to the associated overstressing and crack formation. By providing seepage paths through the wall, crack formation would relieve hydrostatic pressures in affected areas. Loading conditions under which cracking is expected to occur are therefore difficult to predict, due to the complex relationship between crack formation and load distribution. In actuality,

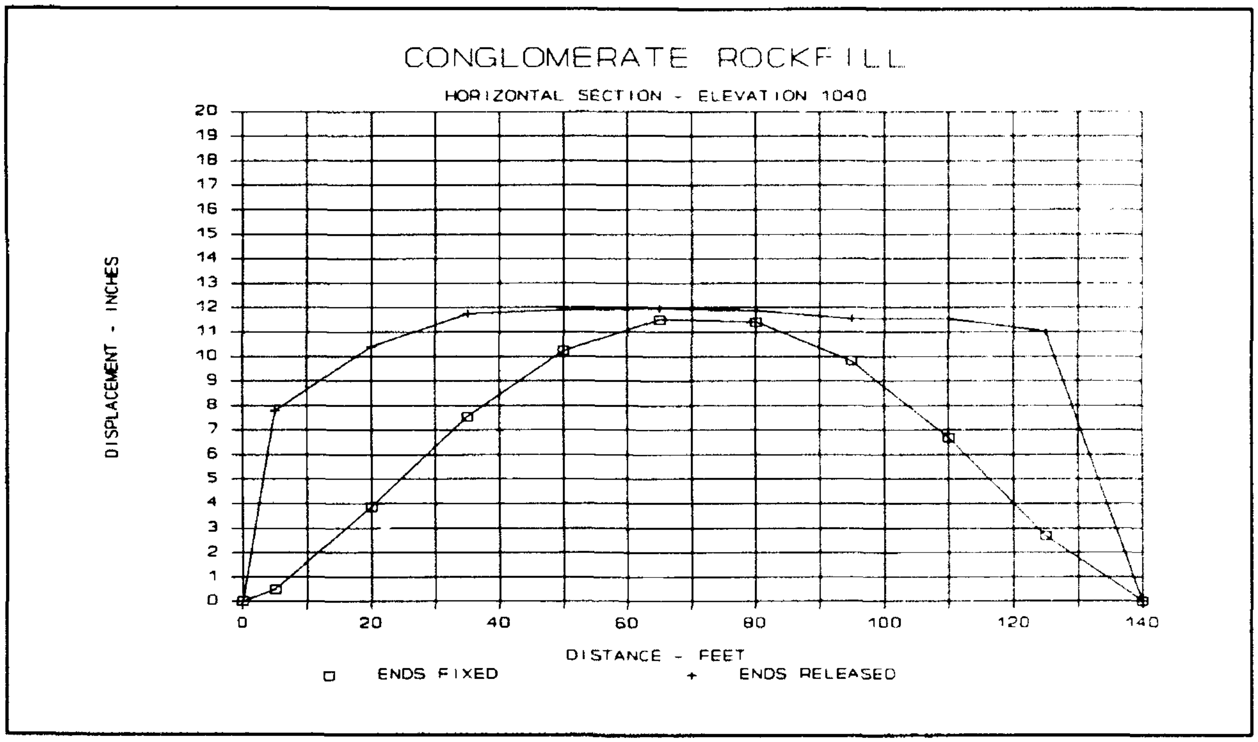

Figure 12. Conglomerate shell, horizontal section of grid before and after end moments released

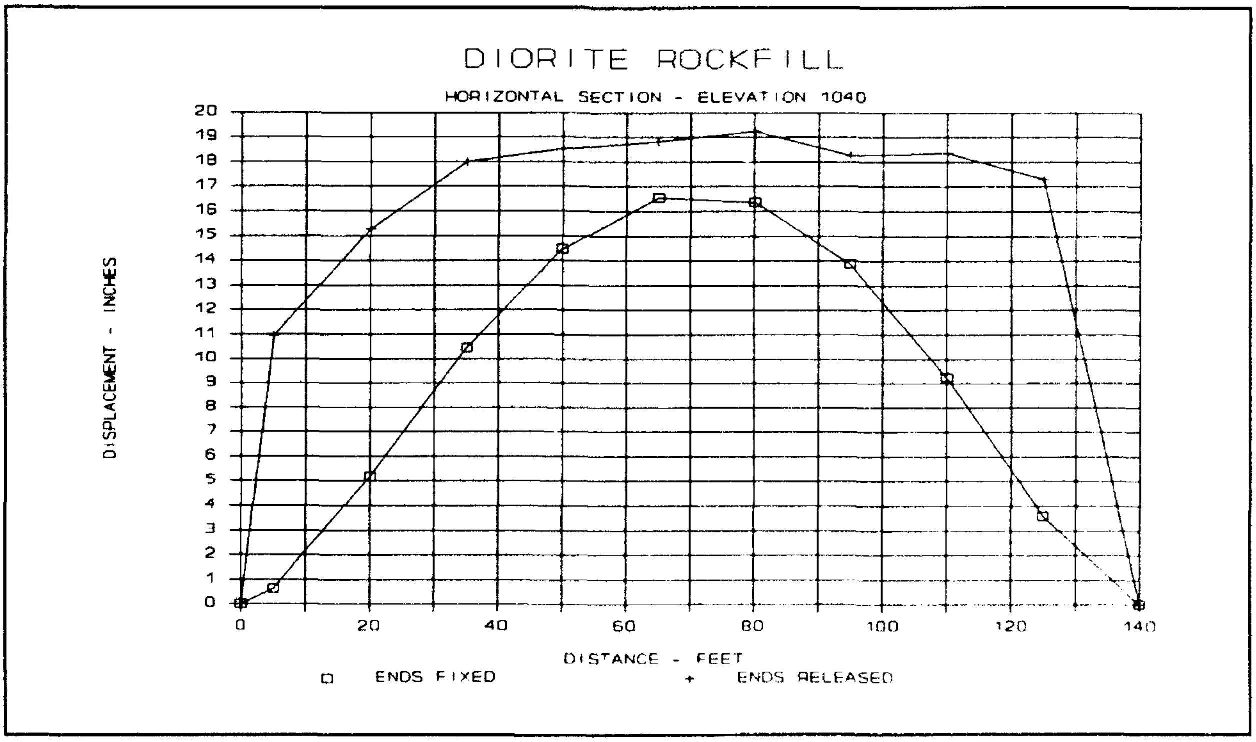

Figure 13. Diorite shell, horizontal section of grid before and after end moments released

seepage around the wall through the moderately faulted rock abutments should provide pressure relief by permitting the formation of a counteracting hydrostatic pressure distribution on the downstream face of the wall. Additional pressure relief may occur due to seepage through joints in the wall itself, although such seepage may be undesirable if it becomes excessive. The loading condition assumed for this investigation does not account for either of these sources of pressure equalization, and therefore may be seen as quite conservative.

The high degree of overstressing revealed by this exercise indicates that crack formation in the wall may occur at an appreciably lower pressure differential than that utilized for this analysis, disallowing the effects of seepage. Due to the previously described relationship between crack formation and pressure distribution, displacements of the magnitude indicated by the analysis are not expected to occur. Displacements would presumably be limited to the point at which seepage due to crack formation serves to relieve the differential pressure at the affected area. Considering the limited moment capacity of the vertical panel joints, this point would potentially occur after only relatively minor displacements have taken place, possibly as little as 1 or 2 in. at the canyon center line.

In spite of these uncertainties which stem from the basic nature of the problem, much can be learned from the results of this investigation. Cracking of the wall, if it occurs, will most likely be along the rock interface, due to the major discontinuity in support stiffness between the soil and rock media. The high ratio of horizontal member stresses to vertical member stresses indicates that cracking would be expected to initially take place along the vertical panel joints at the rock interface at lower pool levels. With increasing pool elevation, cracking would also be expected along horizontal portions of the rock interface at the deepest portion of the wall. Reduction of flexural stresses would require a joint accommodating rotation in these areas, and it is uncertain that a design for such could be achieved which prevents seepage.

The findings of this investigation should not be construed as predicting unsatisfactory performance for the seepage cutoff wall at Mud Mountain Dam. The design of the installation is based largely on similar installations at other sites, experience with which has been acceptable. A monitoring program, consisting of a system of piezometers, survey monuments, and inclinometers, has been included in the installation, which will provide useful information pertaining to loading conditions and deformation response of the wall. The investigation method described provides a means of assessing these data when they become available, making possible a better understanding of the behavior of these structures.

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Seven Oaks Dam Outlet Works
Experiences Influencing Total Quality Design

by
Raymond Dewey, PE[1]

Abstract

Seven Oaks Dam, a 600-ft-high earth-rockfill embankment dam, will be located in the narrow upper Santa Ana River Canyon, approximately 8 miles northeast of Redlands, CA. The project site, between two branches of the San Andreas fault, makes seismic design an essential element of the overall project design.

The construction of Seven Oaks Dam will require 6 years and be completed in July 1997. The outlet works will be constructed in three contracts due to the aggressive construction schedule. The estimated construction cost of the Seven Oaks Dam project is $390 million, of which $46.5 million is for the outlet works.

Introduction

Seven Oaks Dam is a project being designed by the US Army Engineer District, Los Angeles (SPL). In November, 1986, SPL asked the Portand District (NPP) to participate in the design of Seven Oaks Dam. Since that time NPP has been preparing the Seven Oaks Dam outlet portion of the Santa Ana River Mainstem Project General Design Memorandum (GDM) and the Feature Design Memorandum (FDM) for the Seven Oaks Dam outlet works. NPP is presently preparaing the diversion phase plans and specifications. The design has been further complicated by the seismic activity of the region surrounding the project site. The peak ground acceleration is 0.7g for the maximum credible earthquake (MCE) and 0.5g for the maximum probable earthquake (MPE).

During the past year, NPP has emphasized total quality in all that we do. The thrust of the emphasis has been to produce a quality product, to complete the project on schedule, and to accomplish all work within the established budget. How did we achieve quality? What experiences have we had that will help us to better achieve quality next time? Some procedures we followed have enhanced the quality of the Seven Oaks Dam outlet works.

Seven Oaks Dam

Seven Oaks Dam is a part of the Santa Ana River (California) Mainstem Project (Figure 1). This project is designed to provide urban flood protection to the growing communities in Orange, Riverside, and San Bernardino counties. The major components of the project consist of Seven Oaks Dam, delineation of the floodway for a 35-mile stretch between Seven Oaks and Prado Dam, modification of the existing Prado Dam, and modification and improvement of several levees and channels. The estimated project cost is $1.4 billion.

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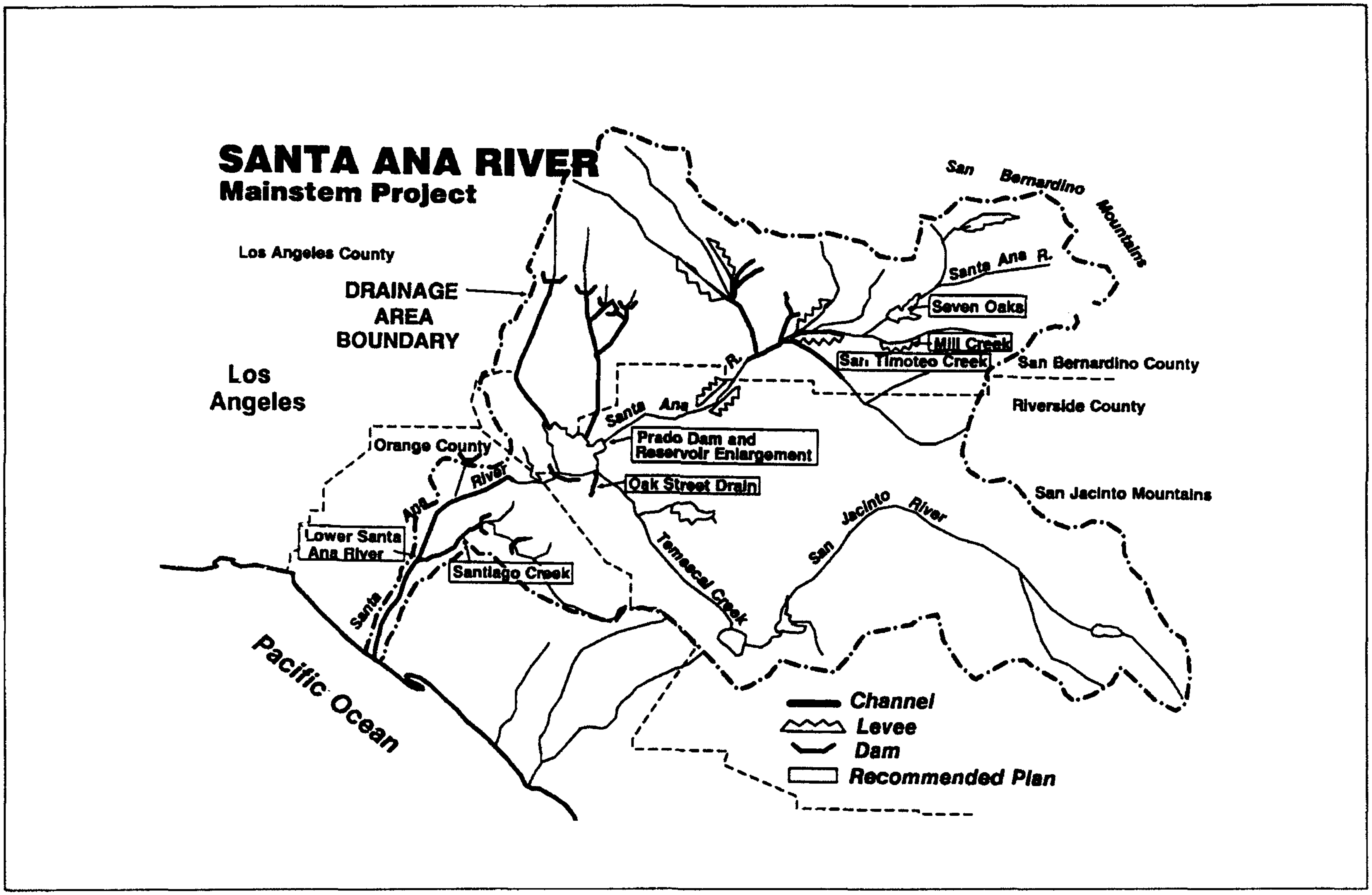

Figure 1. Santa Ana River Mainstem Project Plan

Seven Oaks Dam is located in San Bernardino County, California, approximately 65 miles east of downtown Los Angeles and 8 miles northeast of Redlands. The project site is near the mouth of the narrow upper Santa Ana River canyon. The streambed is at el 2,050. As the Santa Ana River leaves the narrow upper canyon, it flows into a broad valley and winds its way to the Pacific Ocean, south of downtown Los Angeles.

The dominant feature of Seven Oaks Dam is the 600-ft-high earth-rockfill embankment (Figure 2). Through the left abutment (looking downstream) of the embankment dam is the outlet works. Access to the top of the intake structure is by road along the upstream face of the embankment. An access road traverses the top of the dam and provides access to the reservoir area. From the top of the dam, a road on the downstream face of the embankment provides access to the air shaft. Additional roads provide access to the exit channel area and to the spillway. Even farther left is the nonregulated spillway. The outlet works will provide for controlled releases of water from the reservoir during the life of the project.

Outlet Works

The outlet works at Seven Oaks Dam are more complex than would normally be found at a typical project. The outlet works consist of an intake structure, upstream tunnel, gate chamber with vertical air shaft, downstream tunnel, exit channel, valve structure, and plunge pool. They are designed to pass regulated flows ranging from 3 to 7,000 cfs and diversion flows up to 13,000 cfs. After diversion, the intake structure and gate chamber will be modified to their final configuration. The intake structure layout is further complicated by the necessity to handle sediment deposits up to 165 ft high. A small pool will be maintained during 8 or 9 months of the year to encourage the deposition of sediment. During the remaining 3 or 4 months, the reservoir will be dry.

Intake structure

The intake structure (Figure 3) is a 200-ft high structure inclined and anchored against the 4V on 1H excavated rock slope. This structure consists of a 36-ft-diam wet-well and a multilevel withdrawal system (MWS). The upper 35 ft of the intake structure is made up of the maintenance deck and the intake for the wet-well. The wet-well also contains guides for a dewatering bulkhead for maintenance and inspection of the tunnel upstream of the regulating outlet (RO) gates. The MWS consists of a series of ports which will pass water downstream (D/S) when the water surface is below the level of the wet-well intake. These ports will be stop logged as the sediment level increases.

Tunnel and gate chamber

The tunnel connects the intake structure to the exit channel and plunge pool. The gate chamber is located approximately six-tenths of the distance from the intake structure to the exit channel. A 36-in. minimum discharge line (MDL) will be placed beneath the tunnel floor to release flows below 90 cfs. The MDL discharges are regulated by two cone valves located in the valve structure.

The tunnel rock is a moderately hard to hard quartz diorite. It is highly fractured with tight joints. The 1,800-ft exploratory tunnel and core drilling indicate joints, faults, shears, and dikes along the tunnel alignment. The initial tunnel and gate chamber support will be rockbolts and shotcrete. Steel ribsets will also be used at each tunnel portal. The rock parameters used in the design of the tunnel and gate chamber are shown in Table 1.

The upstream tunnel is approximately 1,000 ft long. Except for transitions at the intake structure and the gate chamber, it is an 18-ft-diam reinforced concrete tunnel. During normal operation the upstream tunnel is pressurized.

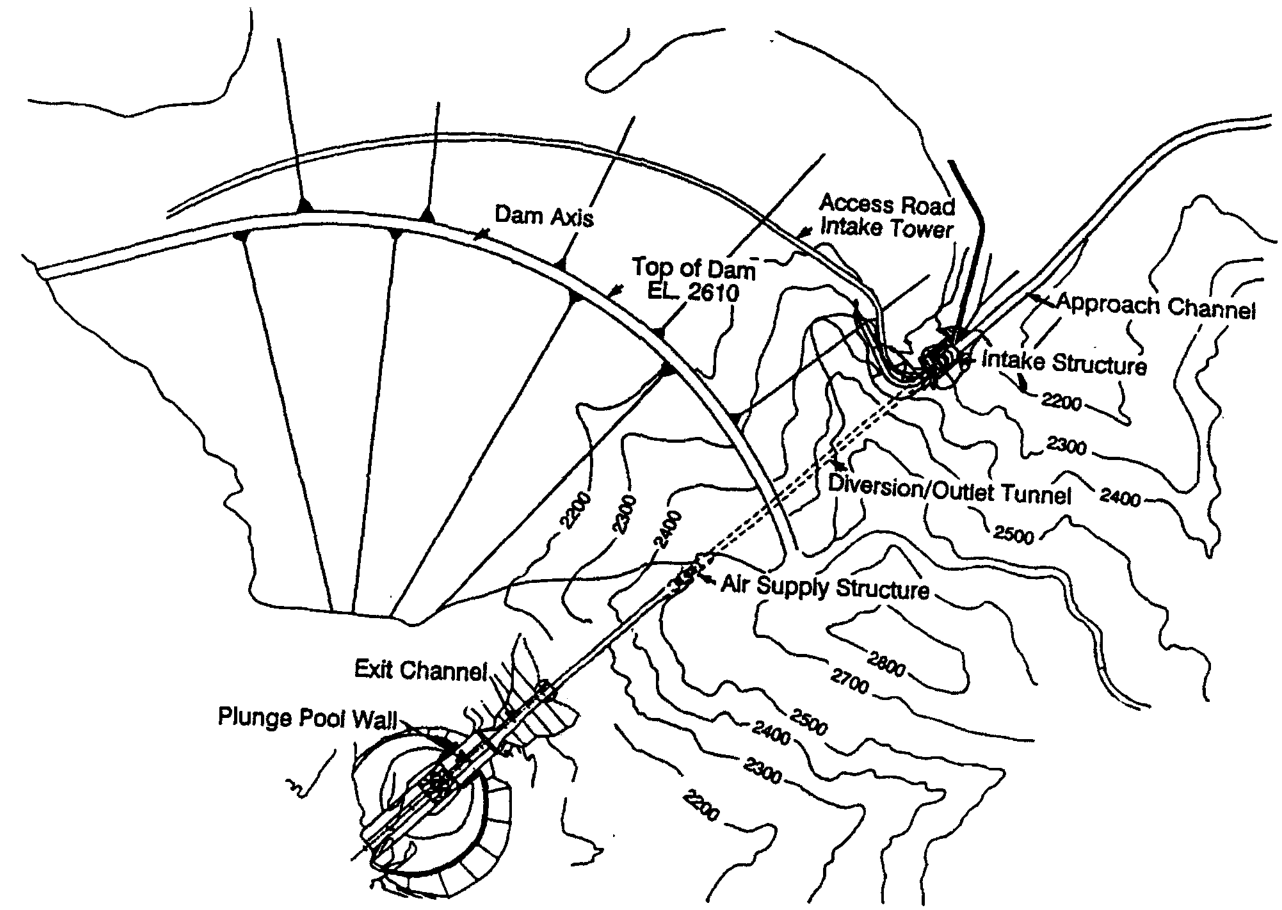

Figure 2. Seven Oaks Dam Outlet Works Plan

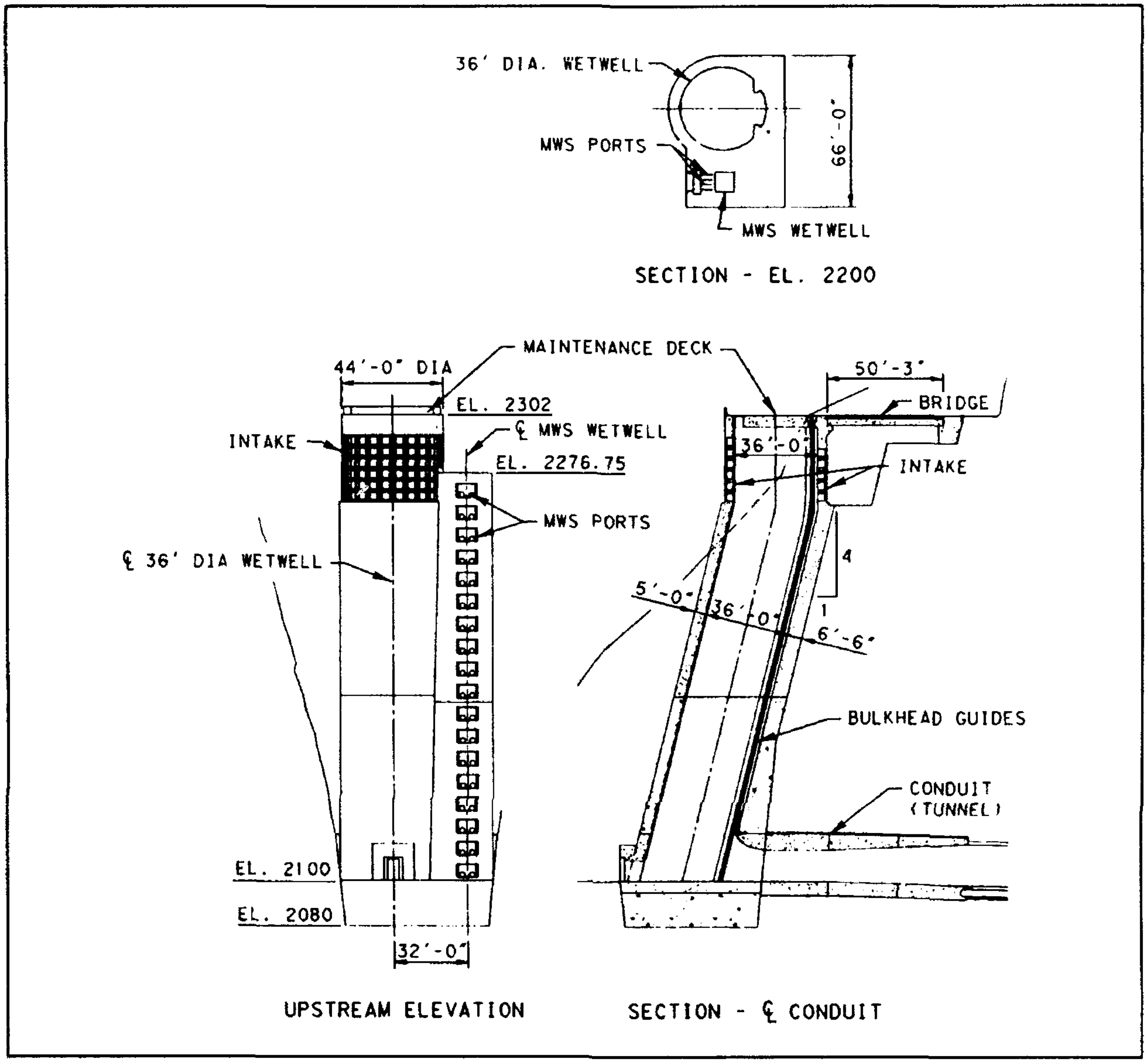

Figure 3. Intake structure

Table 1 **Rock Design Parameters**	
Unit weight	170 pcf
Deformation modulus (average)	0.9×10^6 psi
Unconfined compressive strength	5,000 psi
Friction angle (phi)	35-40 deg
Concrete/rock cohesion (c)	110 psi
Allowable bearing capacity	72 tsf

The reinforced concrete gate chamber contains two RO service gates, two emergency RO gates, a low-flow gate, an emergency low-flow gate, a minimum discharge line ball valve, and other control and maintenance equipment. A set of RO gates (emergency and service) are located on each side of the gate chamber with a set of low-flow gates in the center. The gates are separated by splitter walls. The RO gates are 5 ft wide by 8.5 ft high and the low-flow gates are 2 ft wide by 3.5 ft high. The gate room is a 50-ft-diam by 32-ft-high concrete lined dome. The walls are 4 ft thick. A vertical air shaft penetrates the gate room and bifurcates to provide air downstream of the gates. The air provides a cushion for the high-velocity jet from the RO and low-flow gates. The air shaft will be either 9 or 11 ft in diameter, depending on the excavation method.

The downstream tunnel is approximately 600 ft long. It is an 18-ft-wide by 18.5-ft-high horseshoe-shaped tunnel, except for a transition at the gate chamber. The downstream tunnel is not pressurized. The downstream tunnel has a floor near the springline which provides access to the gate chamber. The floor will be installed after diversion.

Exit channel, valve structure, and plunge pool

The exit channel is 280 ft long. It is a U-shaped channel, 18 ft wide by 14 ft high. At the tunnel portal it is covered with an access structure which contains all the gate and valve controls. The access structure also provides protection from rockfalls. The valve structure is adjacent to the exit channel. It is 21 ft wide by 21 ft long by 14 ft high and houses two cone valves with integral hoods.

The plunge pool is the energy dissipator for outlet flows other than those going through the cone valves. The invert of the exit channel is 35 to 40 ft above the water surface of the plunge pool. The depth of water in the plunge pool is 20 to 30 ft, depending on flows. At most flows the water will impact on the inclined slab which will prevent erosion of the natural fill supporting the exit channel. The size and shape of the plunge pool was developed through hydraulic model testing at the US Army Engineer Waterways Experiment Station (WES).

Quality Design

The NPP design team and managers have been totally committed to quality design of the Seven Oaks Dam outlet works as well as all other projects. Several individual experiences on this project are significant to the achievement of total quality. These experiences involve the relationship of the total design team, the development and evaluation of alternatives, the fine-tuning of design criteria, and the accomplishment of seismic design.

Design team

The relationship of the total design team on Seven Oaks Dam has enhanced the quality of the project. The two things that stand out are the expertise of the design team and the intense coordination.

The expertise of the team members is very important in developing a quality project of this size and magnitude. The NPP design team has designed other high-head outlet works for large projects of the size of Seven Oaks. The high seismicity at the site added an additional complication. Due to a shortage of personnel, NPP sought the services of WES to do finite element (FE) analysis and a consultant to review the seismic design.

The quality of the project has been greatly enhanced due to the significant coordination which has occurred. The project design team is quite large. Overall project management is handled by SPL. NPP has overall design responsibility for the outlet works and SPL is responsible for the main dam and the access roads. The US Army Engineer District, Portland, is also using the services of an architect-engineer (AE), Ebasco Services Incorporated, Bellevue, Washington, for design and WES for FE analysis. SPL used the services of two consultants to look at the project seismicity and to develop a site specific time history. NPP and SPL are also using the services of a design consultant to review the seismic analysis of the outlet works. The design team also contains reviewers from NPP, SPL, the South Pacific Division (SPD), and Headquarters, US Army Corps of Engineers (HQUSACE).
Over the course of several years, we have had many review and team meetings requiring multidesign discipline participation at both NPP and SPL. Additionally, there have been numerous long-distance phone calls to coordinate and gather information. There has been a great willingness to provide the requested information as soon as possible. These relationships have been extremely professional and beneficial.

Alternatives

The Seven Oaks Dam outlet works has been enhanced by the development and evaluation of alternatives. Throughout the GDM and FDM phases, various alternatives were evaluated.

Four alternatives of outlet works with several variables were evaluated during the GDM design phase. These alternatives were as follows: upstream (U/S) control with an inclined intake, central control with a vertical shaft, D/S control with a pressurized conduit, and U/S control with a horizontal gallery. The estimated costs of the alternatives with variables is shown in Table 2. Based on the results of this evaluation, a vertical intake structure with U/S control and a horizontal gallery was selected.

These studies and additional alternative studies during the GDM and FDM design phases provide the basis for the current design and have significantly improved the quality of the project.

Design criteria

The early development and fine-tuning of design criteria is critical to assuring customer needs are met and essential to providing or improving quality. Loads and loading combinations as established in Corps of Engineers guidelines were followed. However, the nature of this project makes adherence to established loading combinations extremely difficult. For example, the normal reservoir elevation shortly after construction is el 2,150 (50-ft head). An extreme flood event could generate a reservoir of el 2,605 (505-ft head). After 50 years, sediment will fill the reservoir to el 2,265. This makes the normal pool about el 2,285 (185-ft head). Various runoff events will create a pool at various elevations between these extremes. What event constitutes a normal event? If the 505-ft pool is an extreme load case with a factor of 0.75, then a 378-ft pool would have a factor of 1.0. Are events that exceed the 378-ft pool infrequent enough to be an extreme load case? The answer is yes. This is a case where the extreme load case controls design. The variability of operation over the life of this project led to the identification of a considerable number of loading combinations. At one time, there were 31 possible loading combinations for the intake structure when all the combinations set forth in Corps of Engineers guidance were considered. To simplify the design, we looked at ways to easily determine the controlling combinations without being overly conservative and yet to develop a design which would always be safe.

Table 2
Estimated Costs (in Millions of Dollars)

Variable	Inclined Access ($)	D/S Access ($)	Shaft ($)	D/S Control ($)
High-level intake	45	32	31	28
Low-level intake	45	30	29	26
High-transition level			33	26
High-head bulkhead			30	27
Steel-lined tunnel				30

Seismic design

The seismic design for the Seven Oaks Dam outlet works has had significant impact on design quality as it relates to the design schedule. The FE analysis has taken approximately 1 year longer to accomplish than was anticipated and, consequently, impacted the design of the intake structure. The seismicity of the site, complexity and uniqueness of the intake structure, and inexperience in seismic analysis of complex structures have all been participatory factors in the time extensions for the analysis.

Seven Oaks Dam is located within seismic Zone 4. The region surrounding Seven Oaks Dam is seismically active due to the interaction of the North American Plate and the Pacific Plate. The relative movement between the plates has formed the San Andreas Fault system. This fault system extends over 700 miles from the Gulf of California to north of San Francisco. The dam site lies between two branches of the San Andreas Fault system. Due to its location, the MCE is considered to be within 1.2 miles of the site and the

MPE within 12 miles. These events have peak horizontal ground accelerations of 0.7g and 0.5g, respectively. A synthetic time history record was developed for the site. After preliminary review of the analysis for the intake structure, a synthetic three-component record was developed.

Since NPP did not have the in-house resource to do the seismic analysis that was required for this project, the seismic analysis was performed by others. NPP and an AE used the results of the analysis for final design. The seismic analysis and design approach was reviewed by a consultant to ensure the safety of the structures.

FE analyses of the tunnel lining, gate chamber, and intake structure were utilized in design. The analysis of the tunnel lining and gate chamber, incorporating response spectrum analysis, have been completed. The analysis of the intake structure is on-going.

A two-dimensional analysis of both the upstream and downstream tunnel lining with surrounding rock was performed (see Figure 4 for downstream tunnel). The seismic stresses in the tunnel lining were small in comparison to the dead and rock loadings.

The gate chamber was a three-dimensional model consisting of 1,399 elements (Figure 5). Due to the size and complexity of the model, only the lining was considered. Elastic springs were used to simulate the effect of the surrounding rock. The model consisted of both plate and brick elements. The output was quite large and a spreadsheet was used to determine the highest thrusts, moments, and shears. Color stress plots were generated, but these were not very useful in designing the concrete

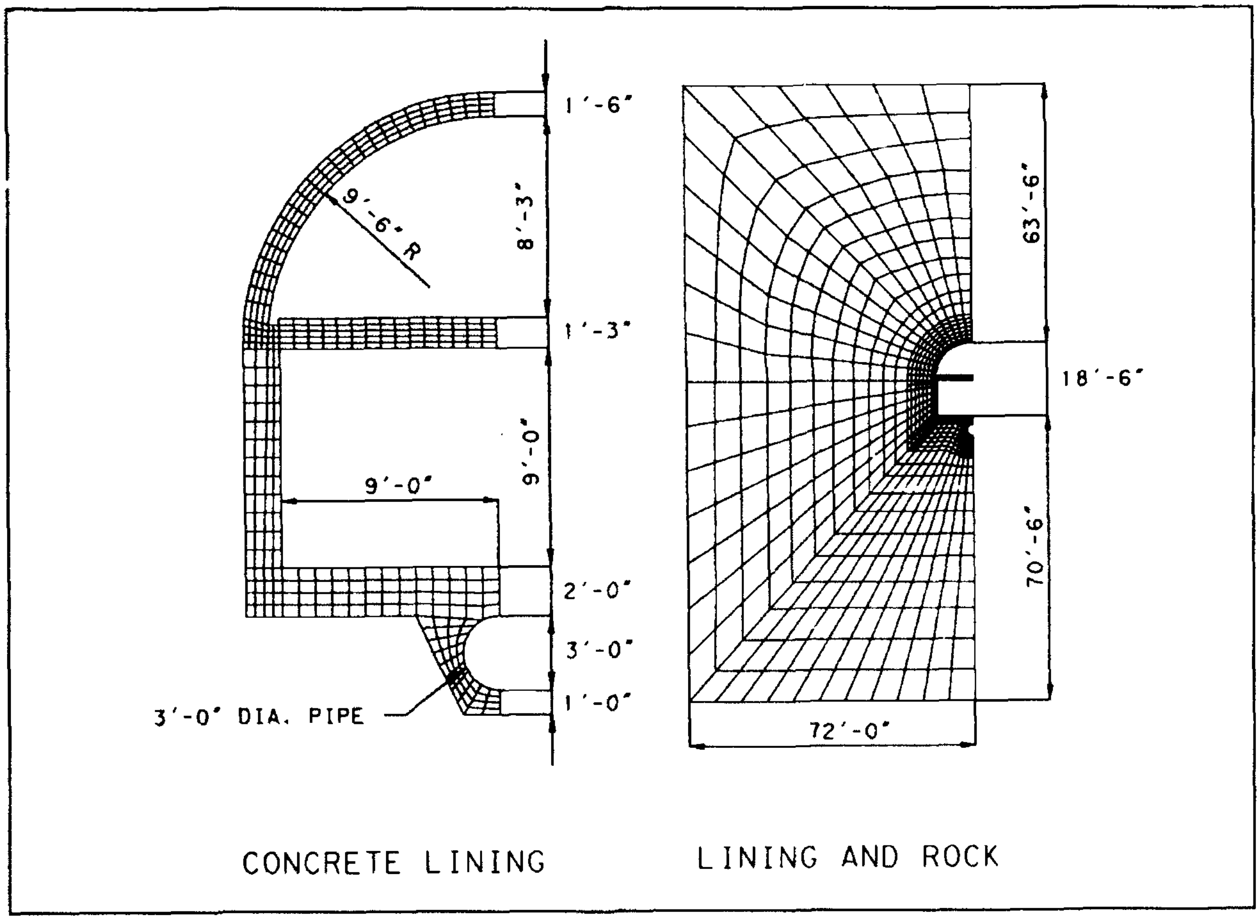

Figure 4. Downstream tunnel FE model

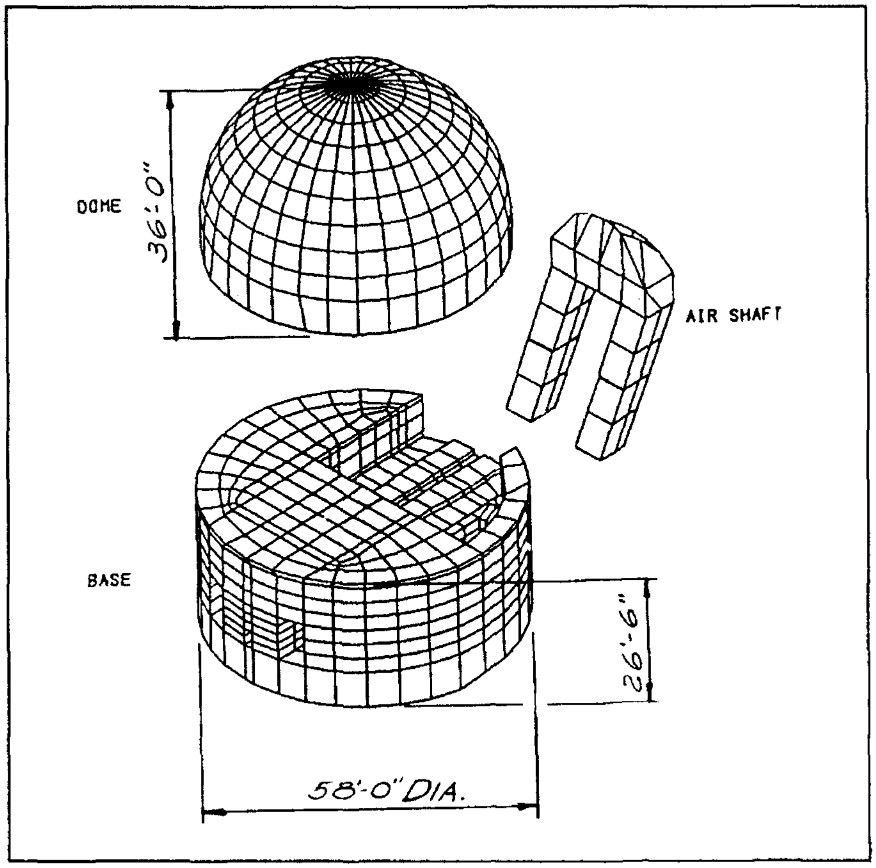

Figure 5. Gate chamber FE model (exploded view)

was applied in the upstream/ downstream direction. This analysis showed that the intake structure and rock did separate and that the impact forces were quickly increasing in magnitude. Approximately 4 seconds of a 70-second record were used in the analysis due to the extreme amount of computer processing time required. The results were not what we had expected; thus we decided to analyze the same model with increased materials properties.

The second analysis was similar to the first except in that the material properties were increased to 3.0 million psi for rock and 4.03 million psi for concrete. The results were very similar to the first analysis. Since further analysis of this model was extremely time-consumming and an anchorage system for the intake structure had been planned, a decision was made to abandon the analyses utilizing contact elements.

where thrust, moment, and shear are usually required.

The seismic analysis of the intake structure has gone through several iterations. The first analysis was a 7,464-element, three-dimensional model of the intake structure and surrounding rock (similar to Figure 6). The analysis was performed using computer code ADINA (Automated Dynamic Incremental Nonlinear Analysis). Special contact elements were used between the concrete and rock. These elements allowed for separation of the concrete from the rock rather than developing tensile stresses. Therefore, the analysis was nonlinear and required a very large amount of computational time. Material properties consisted of a modulus of elasticity of 1.1 million psi for rock and 3.12 million psi for concrete. The seismic loading consisted of a single-component displacement-time history for the MCE, which

For the third analysis, several modifications were made for determining the restraint forces at the sloping concrete/rock interface. These modifications consisted of removal of the contact elements, modifications of the rock grid, and removal of rock beyond the two sides of the structure. Material properties were 3.0 million psi for rock and 4.03 million psi for concrete. The loading consisted of the displacement-time history for the MPE. This analysis was a preliminary step for determining the dynamic and stiffness properties. These properties would then be used in a less complex model.

The model for the fourth analysis consisted of the intake structure and an assemblage of

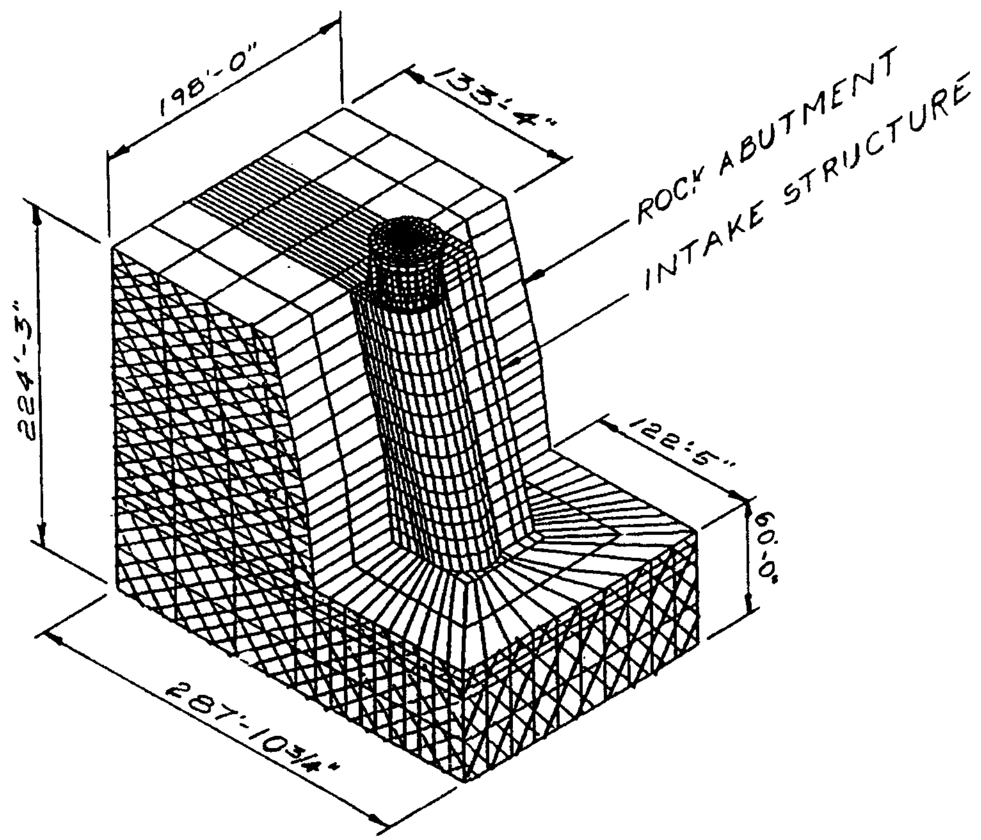

Figure 6. Intake structure FE model (4,843 elements)

truss elements which replaced the rock elements of the previous analyses. The stiffness of the truss elements were modified until the dynamic properties of the model agreed with those in the third analysis. The modal frequencies did not totally agree, but there were reasonable explanations for the variations. The sum of peak tensile stress for the truss elements was 133×10^6 kips for the MCE and 80×10^6 kips for the MPE. When the stiffness of the truss elements was varied, the sum of forces was similar. The ratio of peak mass to peak ground acceleration is 3.0 for the MCE and 2.5 for the MPE. Since these values were what might be expected for a free-standing intake structure, we felt that there was a problem with the analysis.

To resolve the apparent dilemma concerning the analyses, we asked Dr. Jusef Ghanaat, an AE consultant, to review the analyses and provide recommendations. The following were his recommendations:

- Perform linear-elastic analysis of both the structure and rock.
- Reduce the number of elements.
- Use acceleration-time history analysis rather than displacement-time history.
- Perform response spectrum analysis first to get an upper bound for design and to provide a basis for comparison with the subsequent time history results.
- Use dynamic elastic modulus (130 percent of static modulus) for both concrete and rock.

- Use three-component seismic record and consider three-dimensional response of the intake structure due to its nonsymmetrical shape.

The model for the second analysis was revised for the fifth analysis (Figure 6) to reduce the number of elements and use the dynamic elastic modulus for both concrete and rock. This model consisted of 4,843 elements with 15,585 degrees of freedom. A three-component seismic record was utilized. It took several weeks to get this record; therefore, initial analysis utilized a scaled y- and z-direction component. A response spectrum analysis was performed using ADINA. Further study indicated that 20 modes were required to mobilize 90 to 95 percent of the mass. The first three modes involved only the trashrack at the top of the intake structure. The results of this analysis indicated that torsional behavior was evident and important. The sum of the tensile forces at the sloping rock/concrete interface varied as indicated in Table 3. As these results indicate, the transverse component, which only creates tension across the rock/concrete interface by torsion, causes large tensile forces. The hydrodynamic loading also created large forces. The density of water outside the structure was 85 pcf which included the effects of sediment.

Table 3
Sum of Tensile Forces on Rock/Concrete Interface

Component	Tensile Force (kips)	Force Weight
No pool		
Upstream-downstream	16,600	0.26
Transverse	23,300	0.37
Vertical	10,700	0.17
Pool el 2,265, no water inside		
Upstream-downstream	36,700	0.34
Transverse	71,900	0.57
Pool el 2,265, water inside		
Upstream-downstream	40,300	0.33
Transverse	85,600	0.61

Review of the fifth analysis indicated that it was too conservative and the following recommendations were initiated:

- Modify the added mass to account for curvature of the structure.
- Use 62.5 pcf as the density of water. The sediment would consolidate and does not contribute to the dynamic behavior of the structure.
- Use CQC method for direct superposition of the maximum modal response rather than root-sum-square (SRSS). The frequency of modes are too close for use of SRSS.
- Use SRSS method to combine effects of three components.
- Orient components to generate maxi mum response (i.e., apply the main component transverse rather than upstream-downstream).
- Use the three seismic components simultaneously in time-history analyses.

As a result of the sequence of analyses and review, the design schedule and budget were seriously impacted. A large amount of unanticipated effort was expended in developing accurate results for use in design of the intake structure and its anchorage system. We are confident that the quality of the final constructed product will not be adversely impacted; however, there may be an increase in the total cost of design and construction.

This experience has provided several lessons. Seismic design of many civil works features are not covered by design codes or manuals. Seismic analysis by finite element of these structures is extremely complex and not always easily understood. In these situations, we need to consider the following:

- Verification methods. How will the accuracy of the analyses be determined?
- The appropriateness of the mesh. Is it too fine or too coarse?
- The appropriateness of the model.

- Evaluation techniques.
- Use of linear-elastic analysis in lieu of nonlinear analysis.
- Methods for monitoring and maintining the schedule and budget.
- Resources needed to accomplish the analysis.
- The appropriateness of the computer program or code. Will it provide the desired results? Is it appropriate for this application? Will it provide output in a format required for design?

Another lesson from this experience involves the use of AE's. When hiring an AE (or consultant), the quality of the product can be improved by requesting and reviewing the following information:

- Qualifications.
- Experience in the area of interest.
- Experience with similar projects of similar magnitude.
- Who will do the actual work? What is their experience?
- Who will review the work? What is their experience?
- Usage of the software or computer codes on similar projects.
- Reports for similar projects.
- References.

Conclusion

Several features of the Seven Oaks Dam outlet works are very complex and unique. The project is large, and a large number of people in different geographic areas have been involved in the design of the outlet works. The interface of the outlet works with the embankment dam, spillway, and access roads requires coordination and cooperation of the entire Seven Oaks Dam design team. These challanges and interactions have provided learning experiences in the enhancement and management of total quality on this and other projects.

Tainter Gate Analysis

by
David J. Smith[1]

Abstract

The Corps of Engineers apparently is the main designer of large tainter gates; the outside industries use them but generally on a relatively smaller scale. Tainter gates, as designed by the Corps, are a major steel structure, normally 30 ft high by 40 to 50 ft long, and they weigh 30 to 50 tons. There is very little written on the design of tainter gates except material written by Corps of Engineers, the main source of criteria for tainter gate design.

This presentation is a general discussion of the structural design of tainter gates and the lack of criteria. This is prompted by a reanalysis of some old tainter gates, the spillway tainter gates on the main stem dams of the Missouri River.

Engineer Manaul 1110-2-2702 (Headquarters, Department of the Army, 1966), "Design of Spillway Tainter Gates," and the "User's Manual for a Computer Program for Computer-Aided Design/Analysis of Three-Girder Tainter Gates" (Price 1978) do not provide current data and criteria for the design of tainter gates. The major points of question are wave loadings and steel design safety factors.

A tainter gate can be designed today by a structural engineer who is aware of the problems. More research is required to get realistic wave loadings, and geometrical design for ice loads should be defined. The structural design requirements can be written plainly so that the newest engineer on the staff can design a tainter gate.

The Corps is a major source (almost the sole source) of criteria for tainter gate design. Engineer Manual 1110-2-2702 and other publications should be updated to the state of the art.

Introduction

Why evaluate tainter gates?

The evaluation of the structural strength of tainter gates (Figure 1) was identified as a necessary part of the dam safety program. Many tainter gates designed before 1960 did not have criteria or loading conditions that are presently defined. Many times trunnion friction has been ignored. Tainter gates designed before 1960 did not apply the more sophisticated equations for beam-columns and buckling of compression members later developed by the American Institute of Steel Construction (AISC). Wave loads are not fully defined even today.

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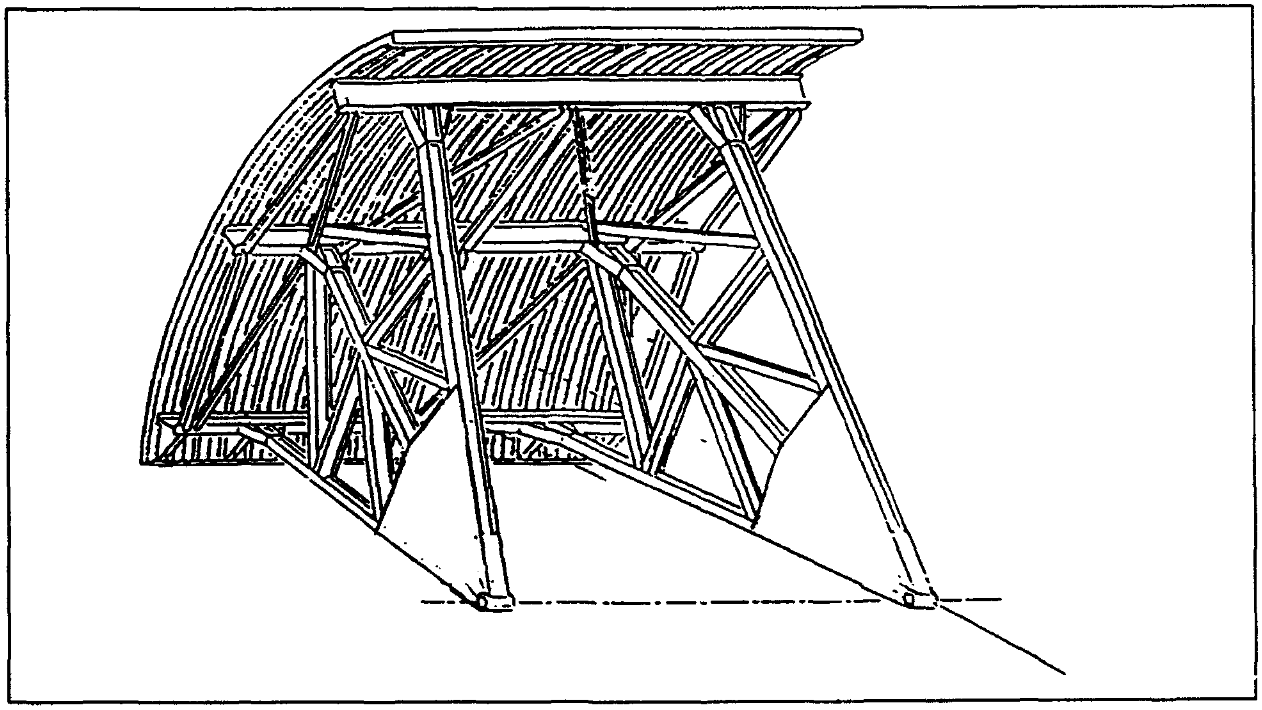

Figure 1. Tainter gate

What is a tainter gate?

A tainter gate is a circular or, more accurately, a segment of a circle which retains or holds back water (Figure 2). The gate has a *framing system* which can sit on a sill, but the hydrostatic load is mainly supported by pivot pins or trunnion pin (Figure 3). The trunnion pin is anchored into a massive concrete pier.

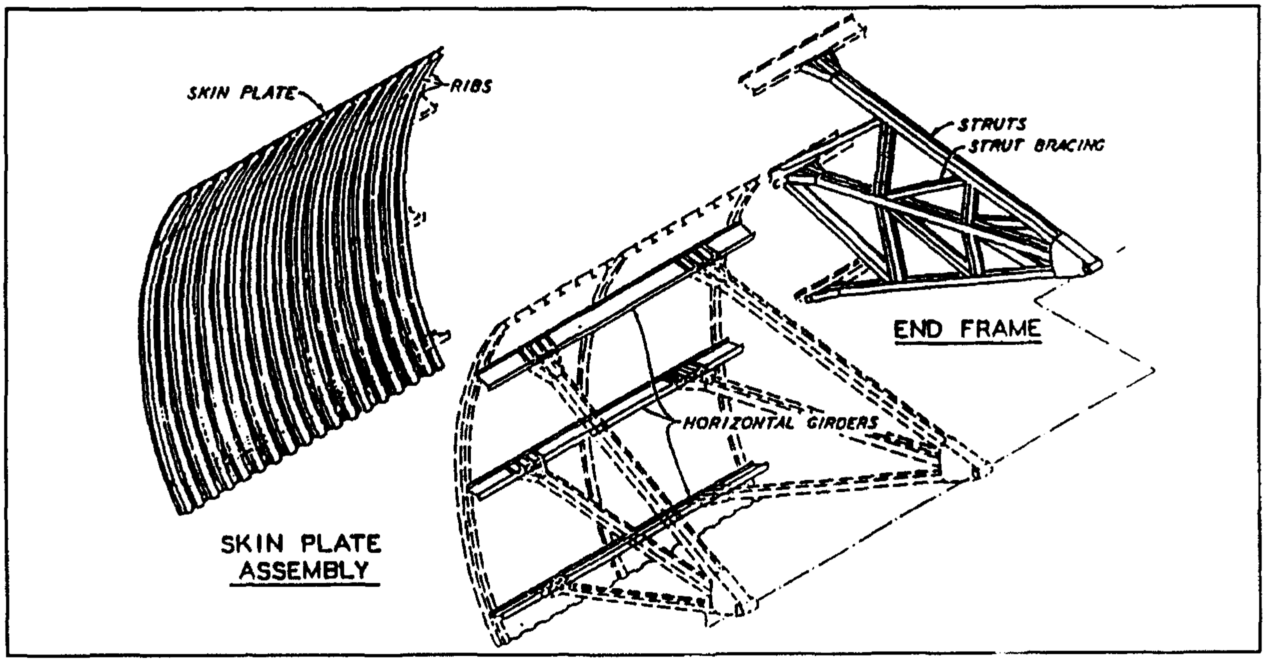

Figure 2. Primary structural members

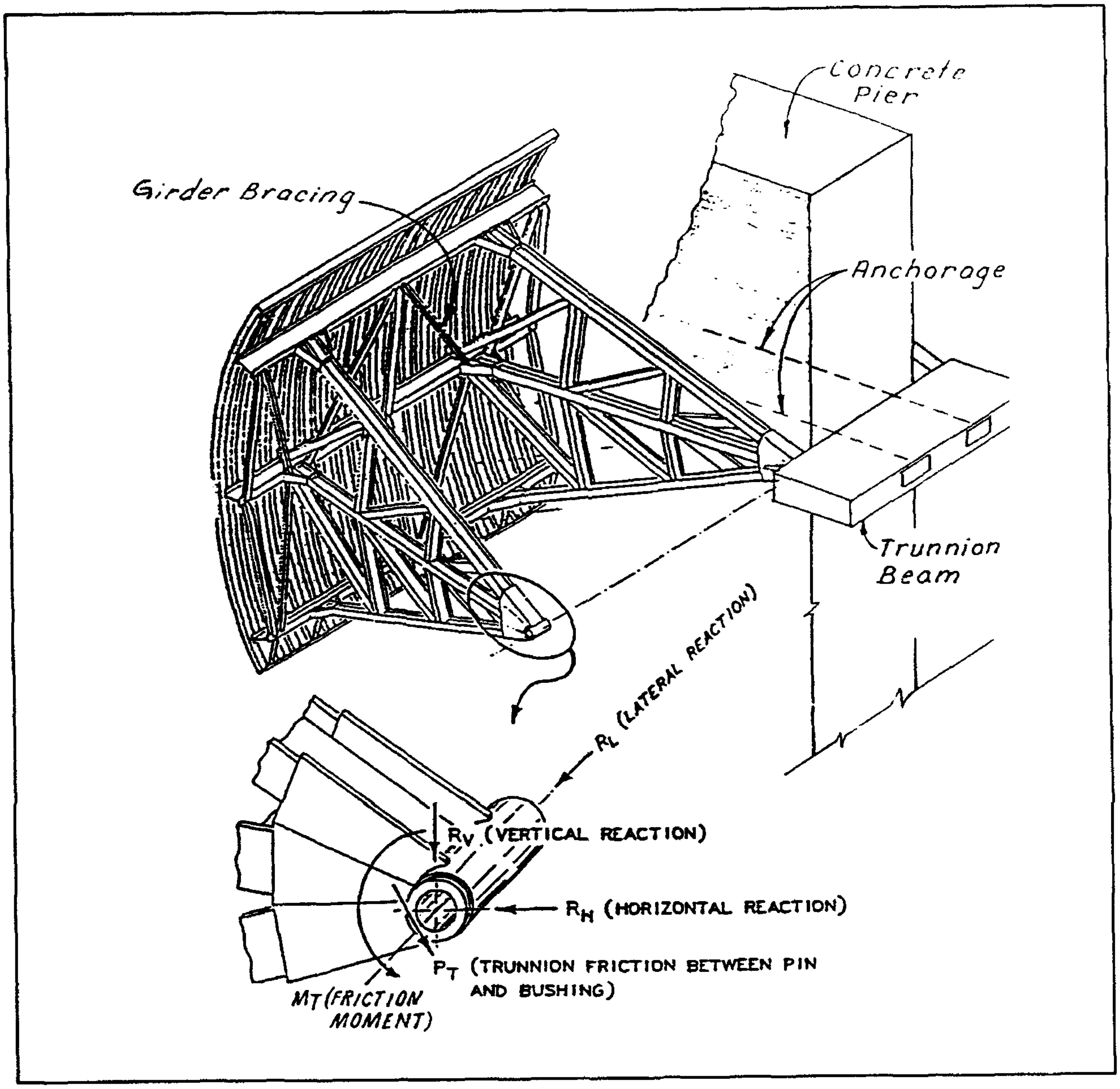

Figure 3. Trunnion reactions

Why a circular shape?

The reason for a circular shape (Figure 4) is that fluid pressure acts normal to the surface of contact and the lines of force pass through the trunnion pin. This means that water pressure is not a consideration when computing Sill Reactions nor when computing cable tension for lifting the gate.

Design and cost

Tainter gates are generally designed to minimize the quantity of structural material and, thus, have minimum factors of safety for the material strength. They weight about 25 tons and cost between 3 and 4 million dollars each.

Sources of design data

The engineering manual for tainter gates is EM 1110-2-2702, "Design of Spillway Tainter Gates," (Headquarters, Department of the Army, 1966).

The US Army Engineer Waterways Experiment Station (USAEWES) published a comprehensive theory of tainter gate design in the form of a manual for a computer program, "A Computer Program for Computer-Aided Design/ Analysis of Three-Girder Tainter Gates," (Price 1978). The program includes wave load equations from the Sainflou Theory which was published as part of *Shore Protection Manual*, (USAEWES 1984).

Loads

Using EM 110-2-2702 forces the designer to start from scratch to define load cases. The computer program manual defines 14 load cases (Table 1).

There are two basic loading conditions considered for design: hydrostatic load and hydrostatic with wave load. Ice loading is considered a special case of hydrostatic load. Each loading was considered with the gate resting on the sill and again with the gate being lifted (i.e. suspended by the cables or chains).

Hydrostatic loads

Water to the top of a gate is the maximum basic hydrostatic load condition. This load is the most logical starting point and provides a datum from which to evaluate loading conditions which may be of interest but may not need a complete set of computations and computer runs.

Wave loads

Many early designs did not include wave loads on the tainter gates. In some cases the designers have used surcharge loads, we assume to account for wave action. The Sainflou Theory wave pressures and reflected heights are generated by the tainter gate computer program. However, these had to be revised because the reflection factor was incorrect for water to the top of a circular gate. New reflection data was reported in "Wave Reflection at Tainter Gates; Hydraulic Model Investigation" (Smith, Earnest R., (in preparation)). The significant wave heights are computed in accordance with methods detailed in ETL 1110-2-305. MORE WAVE REFLECTION STUDIES ARE NEEDED!

Cable radial force

Tainter gates are lifted by using a hoist mechanism above with a cable (or chain) bearing on the skin plate. When the tainter gate is lifted there is tension in the cable, and, since the cable is essentially wrapped around the gate, a radial force is induced. The radial force is not identified in EM 1110-2-2702. Cable induced radial force affects the strut design.

Dead load

The dead load reaction of the gate resting on the sill causes a significant additional load on the ribs/skin plate and the bottom frame (struts and girder). One computation showed the load on the bottom girder was increased 12 percent over the suspended gate load case.

Ice loads

Ice loads are 5 kips/lin ft according to EM 1110-2-2702. Stability of the gate is a consideration when you have a concentrated horizontal line load. Ice loads are often not a problem when reservoir elevations are lower during winter months, and a lower ice elevation is acceptable.

Unsymmetrical loading

Failure of one side of the hoist mechanism or cable would cause unsymmetrical loading. Such loads do not have a system for redistribution of stress except for the skin plate. Girder flange bracing can be used in such a way to help this problem.

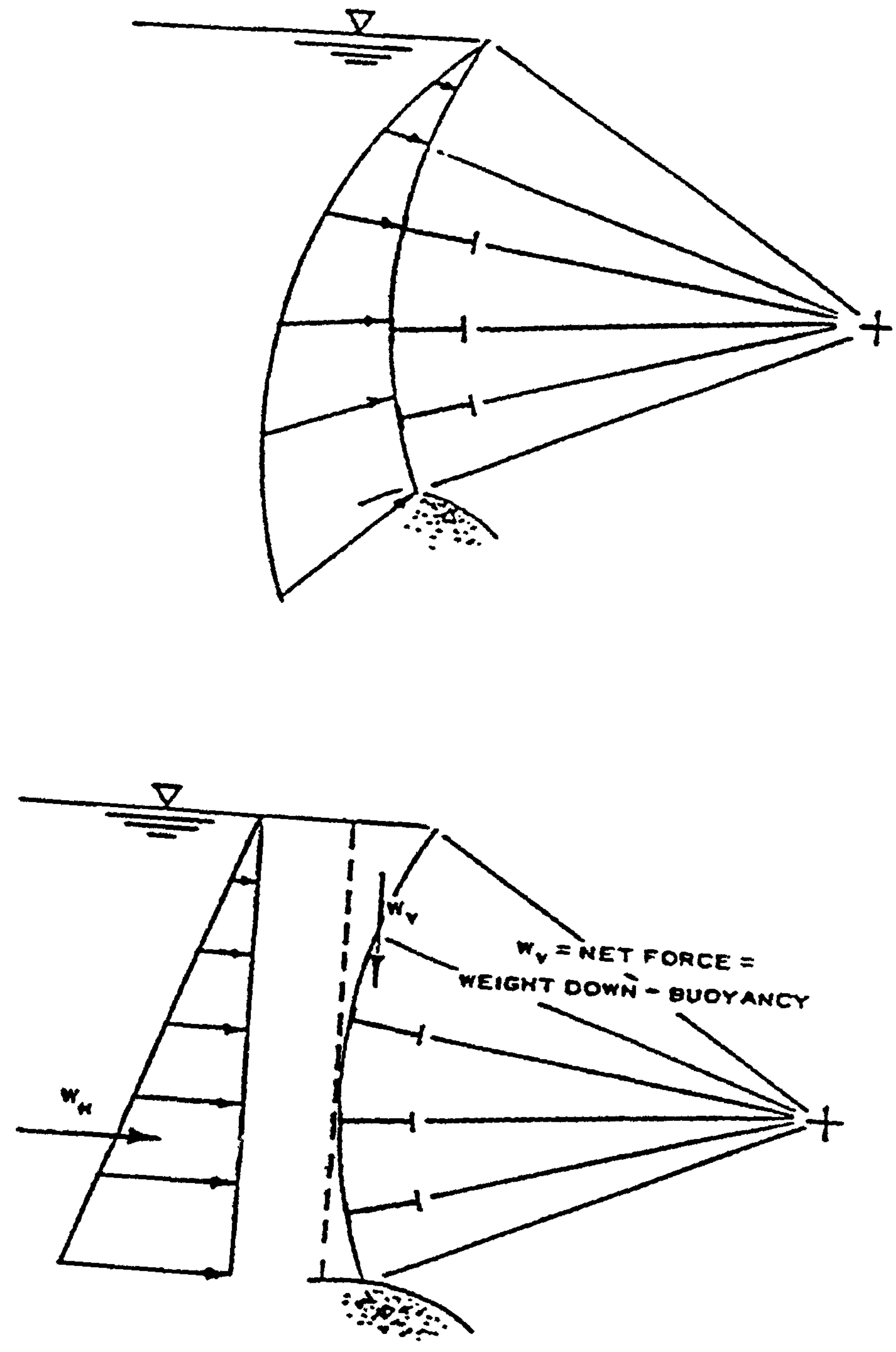

Figure 4. Hydrostatic pressure

Table 1 Load Case Definitions	
Load Case	**Description**
1	Gate resting on sill. Headwater and tailwater. Usually load group I (0SF12 = 1.0).
2	Like case 1 except gate on two cables instead of sill.
3	Gate resting on sill. Headwater and tailwater. Data for Sainflou Theory nonbreaking H_1 wave. Usually load group II (0SF34 = 1.3333).
4	Like case 3 except gate on two cables instead of sill.
5	Gate resting on sill. Headwater and tailwater. Sustained line-load force at any elevation (ice default = 5,000 lb/ft). Usually load group I (0SF56 = 1.0).
6	Like case 5 except gate on two cables instead of sill.
7	Gate resting on sill. Headwater and tailwater. Impact line-load force at any elevation (debris impact = 10,000 lb/ft). Data for Sainflou Theory nonbreaking H_1 wave. Usually load group II (0SF78 = 1.3333).
8	Like case 5 except gate on two cables instead of sill.
9	Gate on one cable (sidesway limited). Headwater and tailwater. Usually 50 percent overstress (0SF9 = 1.5).
10	Gate on two cables, sides of gate jammed against monolith walls, hoist at stall pull. Headwater and tailwater. Usually load group II (0SF10 = 1.3333).
11	Gate resting on sill. Headwater and tailwater. Earthquake added-pressure values (up to 10 values). Usually load group II (0SFEQ = 1.3333).
12	Like case 11 except gate on two cables instead of sill.
13	Gate on two cables with hoist stall pull. Gate rotated up to stops. Gate up out of water. Usually load group II (0SF13 = 1.3333).
14	A special load case not normally used: From 2 to 10 hydrostatic pressure values, at specified elevations on the gate. No headwater or tailwater. No cables or sill reaction. Usually load group I (0SF14 = 1.0).

Analysis

Computer programs

The tainter gates are usually analyzed using computer programs. The ones most readily available for our use are:

- "Computer-Aided Design/Analysis of Three-Girder Tainter Gates," WES LIB/ Corps X0051 (Price 1978). The tainter gate program is an excellent design program. The program is set up for three girders only. Normally, this is the most economical system. However, analysis is not quite as efficient. It does not select the most critical condition for trunnion strut failure mode. The first failure condition encountered sends you to a redesign mode. Also, the program will not recognize a lack of strut bracing or a poor configuration of strut bracing.

- "Computer Program with Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)," (Hartman and Jobst 1983).
- STAAD-III/ISDS (Structural Analysis And Design/Integrated Structural Design System) is a proprietary computer program of Research Engineers, Inc., Marlton, NJ. This is a finite element code that has beam elements and plain strain plate elements. Three-dimensional analysis is allowed in this program but was not considered necessary for the tainter gates. The beam elements can be checked against the AISC Code (AISC 1989) within this program. It was discovered that using 0.8333 for the interaction maximum requirement is in agreement with EM-1110-1-2101, "Working Stresses for Structural Design" (Headquarters, Department of the Army, 1963), stress requirements for hydraulic structures.

Design

EM 1110-2-2702 says "This manual outlines the procedures to be followed in developing designs for tainter gates." Unfortunately, this is not true. The manual provides good data but lacks in procedural guidance.

However, we do have excellent guidance in the three-girder tainter gate program (Price 1978). This covers almost all the theory and considerations needed to understand the design of tainter gates. This computer program does most of the laborious computations. The as-built analysis ends with the first criteria failure, not necessarily the worst condition. Bracing for the struts, i.e. the end frame, is assumed to be good, but it is not designed.

Skin plate

This is well covered in EM 1110-2-2702 and by the tainter gate program (Price 1978).

Skin plate and ribs

Figures 2 and 5. This is covered by the tainter gate program (Price 1978).

Horizontal frames

See Figures 2 and 5. The same tainter gate program just referenced. This is easily analyzed by most any frame analysis program.

End frame (struts & bracing)

This is illustrated by Figures 2 and 6. Figures 6a and 6b are taken from the tainter gate program manual (Price 1978) and show two and three bracing points. These are to reduce length or rather the slenderness (l/r) ratio of the braces.

Figures 6c and 6d show braces at midpoint of the weak axis of a W-beam which usually results in equal l/r ratios in the strong and the weak axis directions. These were developed as a variation of those shown in Figures 6a and 6b. Notice the large trunnion hub plate. The only "criteria" for trunnion hub plates was a small print note on a sketch in the computer manual (Price 1978) that said "Default length 7 feet." This appears to translate to about 20 to 25 percent of the length of the strut. This seems logical when a triangular shaped moment diagram would be drawn for a simple beam due to a moment applied at a support. Then because you are actually working with a truss, the bending effect apparently resolves to variations in axial load.

Figure 6e is a different configuration that is shown in the EM 1110-2-2702. The mathematical model for this assumes that all joints are pined joints and thus the total effect of the trunnion pin friction carried by the internal bracing ending at the cable support. Using the statical method of sections you can cut a section at any plain that cuts only one bracing member. Then, taking moments at the trunnion pin, the force in the bracing member can be computed......This appears to be an incorrect mathematical model.

Design for overtopping

Tainter gates are normally not designed for overtopping.

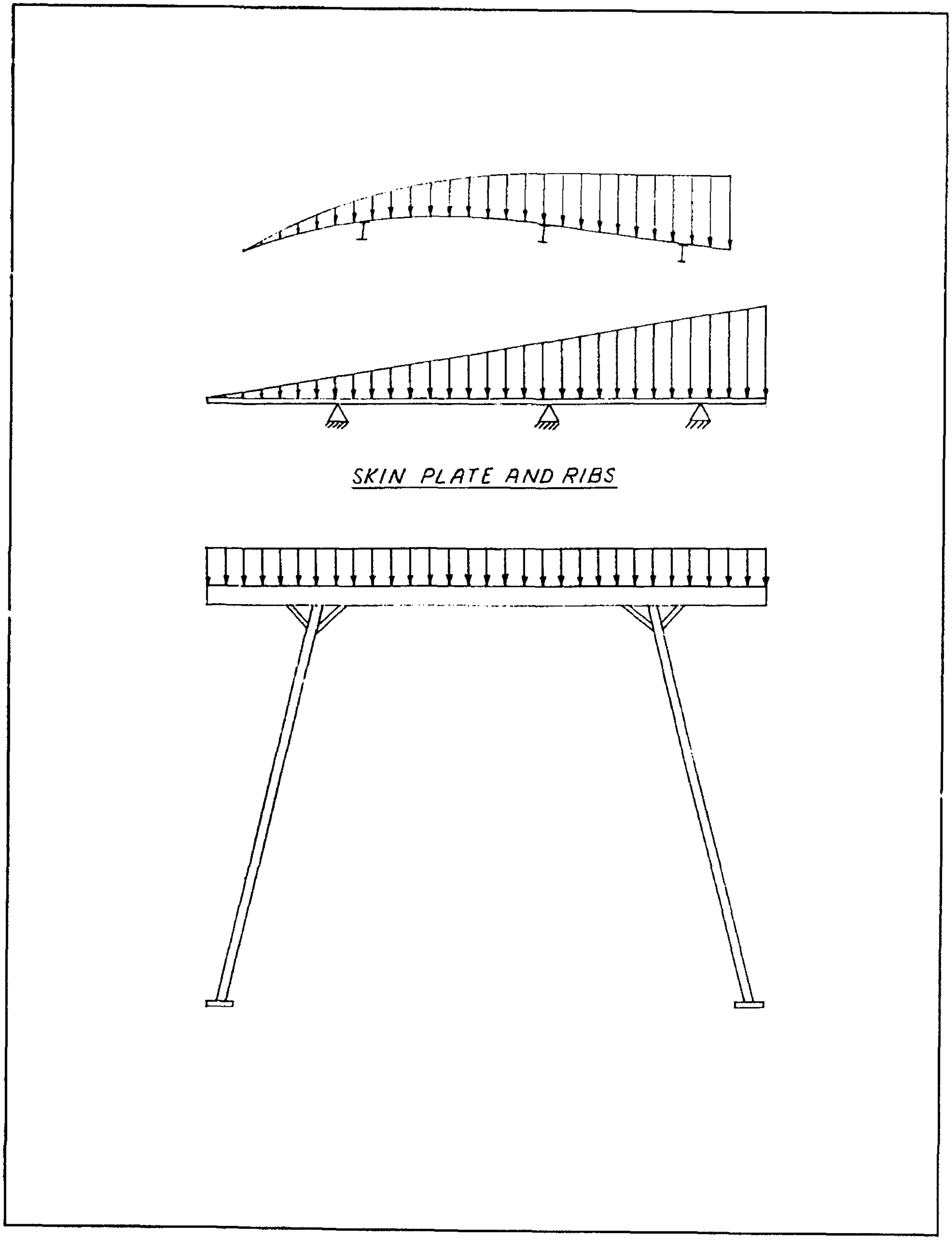

Figure 5. Horizontal frames

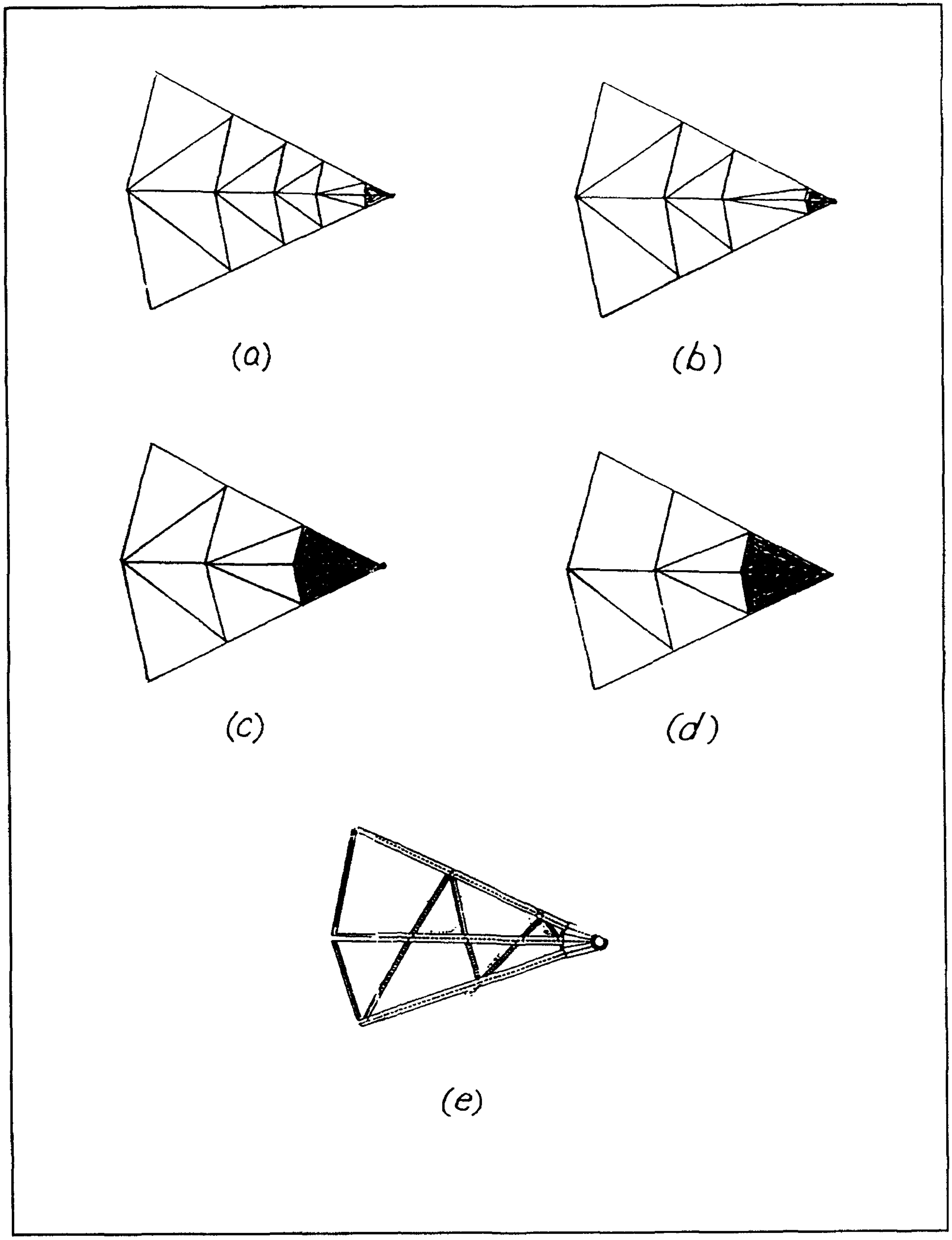

Figure 6. End frame configurations

Overtopping is a very special design condition and should not be considered except when there are very special reasons and justification for the cost.

Project office people seem to think it is OK to allow water to overtop the tainter gates during "Crisis Exercises." PLEASE OPEN THE GATES!

Computations for a case with 3 ft of overtopping resulted in total anchorage failure. Theoretically, the gates went downstream. PLEASE OPEN THE GATES!

Conclusions

Since we do not design tainter gates every day, we need:

- An updated engineer manual on tainter gates.
- Wave studies that apply to tainter gates.
- Criteria that really.......*outlines the procedures* to be followed in developing designs for tainter gates.

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Submersible Tainter Gate Addition to Peoria and LaGrange Wicket Dams

by
David R. Wehrley, PE[1]

Abstract

Peoria and LaGrange Dams on the Illinois Waterway are two of the few wicket dams remaining in the United States. The wicket dams provide for open-pass navigation, however, they are hazardous to operate and have no ice or debris passage capability. To improve the safety and efficiency of dam operation, a section of each of the wicket dams was modified by the addition of a submersible tainter gate and appurtenant structures. This paper addresses the design considerations and construction experience for these projects which were completed in the fall of 1990 with a total construction cost of about $20 million. An internally braced single-wall sheet-pile cofferdam designed by the US Army Engineer District, Rock Island, was constructed to dewater a 112 by 117-ft dam section at each site. This design was determined to be more economical than a cellular cofferdam. Model studies were made to size the scour protection and ensure navigational safety. The gate was positioned adjacent to the lockwall where it would have the most benefits, however, scour had to be prevented to preserve lockwall stability. Double pocket wheels at each end of the gate hoist the round link chain connected to the gate.

Introduction

General project information

LaGrange Lock and Dam are located 80.2 miles (129.1 km) above the mouth of the Illinois River, 8 miles (12.9 km) downstream of Beardstown, IL. Peoria Lock and Dam are 77.5 miles (124.7 km) upstream of LaGrange Lock and Dam, 2 miles downstream of Peoria, IL. These two facilities are the farthest downstream locks and dams on the Illinois Waterway which link Lake Michigan at Chicago to the Mississippi River at Grafton, IL, approximately 38 miles (61.2 km) upstream of St. Louis. The Illinois Waterway project and its maintenance were authorized by the River and Harbor Act of July 1930; the Peoria and LaGrange projects were put into operation in 1939. The dams at Peoria and LaGrange are wicket dams, two of the few remaining in the United States. The wicket dams (each about 540 ft wide) allow tows to bypass the locks in open-pass navigation when flows are sufficient to lower all of the wickets. These open-pass conditions are present on an average of 40 and 47 percent of a year at Peoria and LaGrange, respectively. Each wicket is 3 ft, 9 in. (1.14 m) wide and set with approximately 3-in. (7.6 cm) gaps between adjacent wickets. The LaGrange wickets are 14 ft, 11 in. (4.55 m) long and dam a vertical height of 14.0 ft

[1] Project Engineer, Design Branch, US Army Engineer District, Rock Island; Rock Island, IL.

(4.26 m) when in the raised position. The Peoria wickets are 16 ft, 5 in. (5.00 m) long and dam a vertical height of 15.4 ft (4.69 m) when in the raised position. The maximum head on the LaGrange and Peoria Dams are 10 and 11 ft, respectively. Normal upper pool is el 440.0 at Peoria and el 429.0 at LaGrange.

Dam Improvements made - submersible tainter gates

By separate construction contracts initiated in September 1987, 26 wickets at each of the subject dams adjacent to the lockwall were replaced with one 84-ft-wide submersible tainter gate and appurtenant piers, gate sill and apron, and service bridge. Details of the dam improvement are shown in Figures 1, 2, and 3. The clear distance between piers is 76 ft (23.1 m) and the out-to-out pier distance is 104 ft (31.7 m). The gates can be submerged to 8 ft (2.4 m) below normal pool, raised clear of a 500-year flood, or set at any in-between position.

Because Corps submersible gates on the Ohio River have historically experienced severe vibrations [1], the Rock Island District was advised by higher authority to complete a model study examining gate vibrations. Later in this paper, the parameters within which submersible tainter gates will operate effectively will be hypothesized.

Need For Dam Improvement

Functional wicket dam limitations

Each wicket has only two stationary positions - raised or lowered. With low river flows, all of the wickets must be raised and the water passes through the gaps between wickets. Since the normal pool elevation is at the top of the raised wickets, ice and debris cannot pass through the wicket dam while the wickets are raised. Each dam has several flip-top, hinged wickets originally intended to pass ice and debris. Although the flip-top wickets with the top 3 ft lowered maintain the pool better than fully lowered wickets, the flip-top wickets cannot sufficiently pass heavy ice flows. The dams had no other section for passing ice or debris. Backed-up ice can cause several problems, some of them operational as discussed. The ice can hinder the movement of tows and slow lockages. Frequently during the winter, ice lockages are required to move large quantities of ice through while tows wait for a clear lock.

Flow regulation with the wickets is undesirable, but was necessary before the tainter gate was added. The lowered wickets pass a column of water that causes severe scouring action. In addition, the wickets cannot be efficiently raised or lowered as flows change. The response time to a needed change in wicket settings is slow.

Operational wicket dam limitations and hazards

Every time the wickets are raised or lowered, a five-person crew, a push boat, and maneuver boat (equipped with a boom and steam-powered winches) must be mobilized. Dam operations tie up the same crew responsible for lock operation.

Much of the operation of raising or lowering wickets is slow and hazardous manual work. Heavy pole hooks are manually attached to the wickets with the crew working close to the edge of the maneuver boat and nearby flowing water. To raise a lowered wicket the crew must extend the pole hook through flowing water to fish about for the breech strap (attached to lower end of each wicket) which is often buried in sediment.

Raising and lowering the wickets with ice in the pool is hazardous. The ice pressure has capsized the work boat and forced the maneuver boat through the dam on several occasions. In addition, these operations are more difficult with a high head on the dam which is the case for the first 20 wickets lowered or the last 20 wickets raised. Associated with the high head is high velocity, turbulent flow, and a high pressure on the raised wickets that must be overcome.

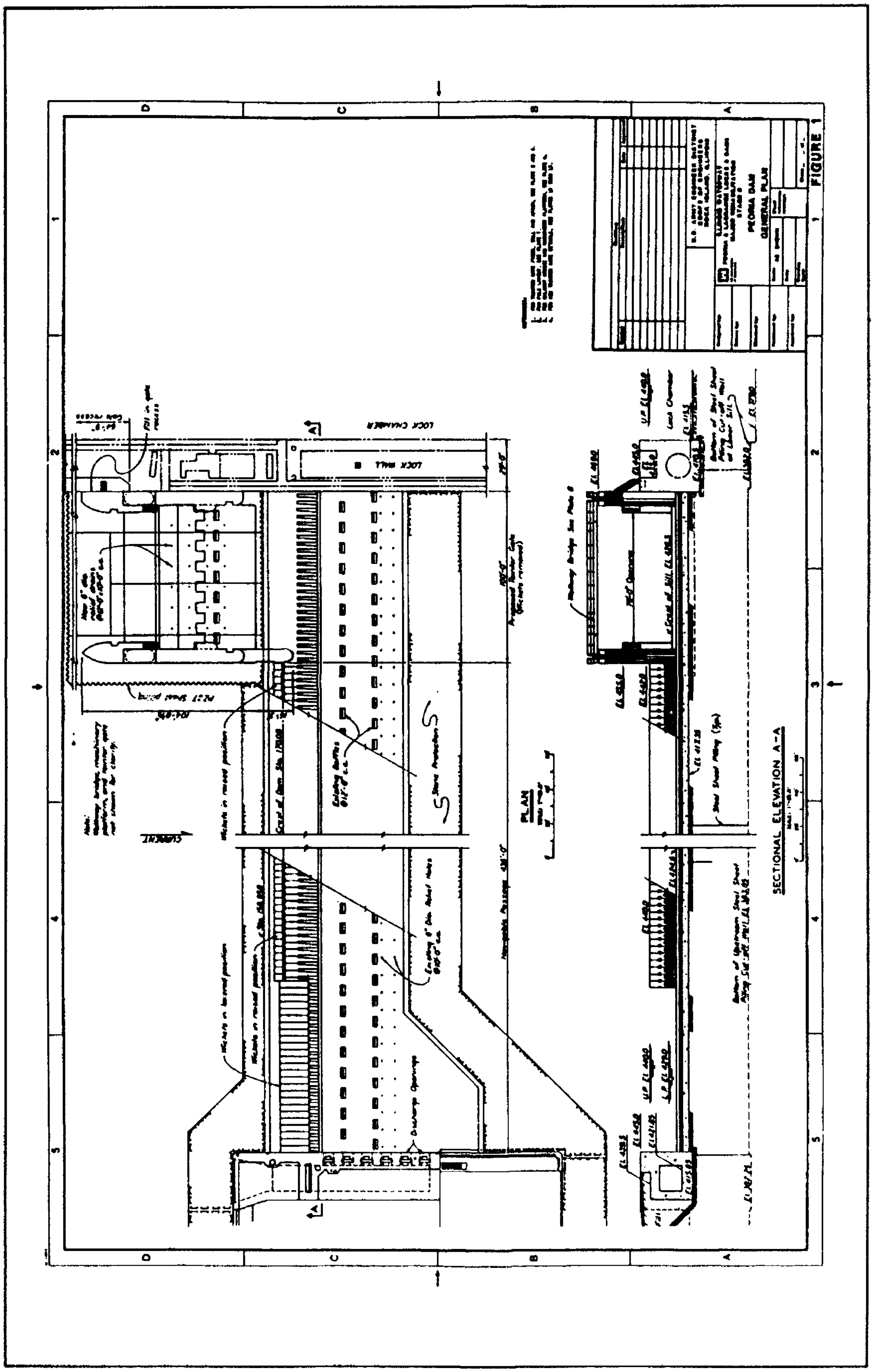

Figure 1. Peoria Dam—general plan

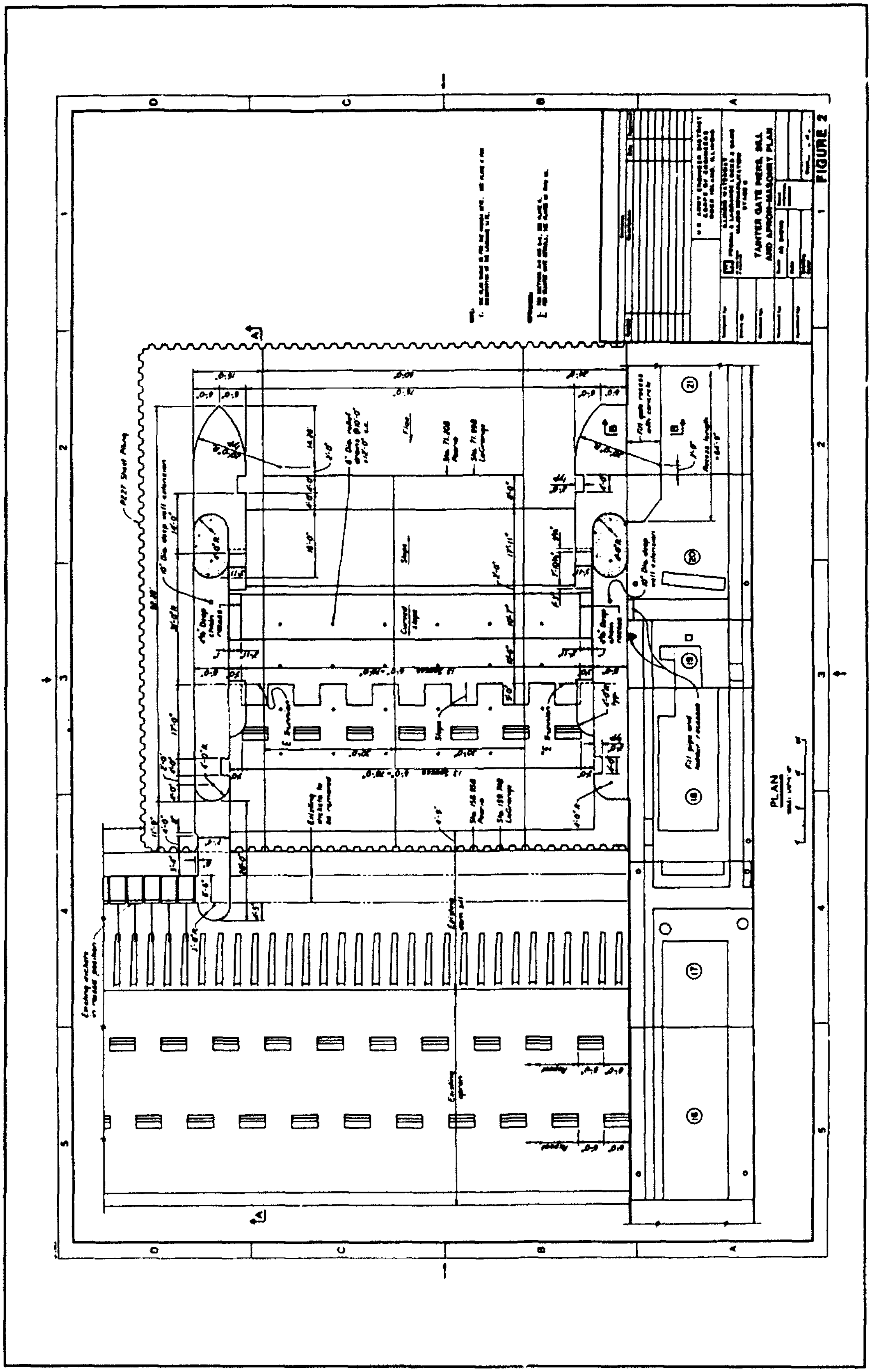

Figure 2. Peoria Lock and Dam—tainter gate piers, sill, and apron-masonry plan

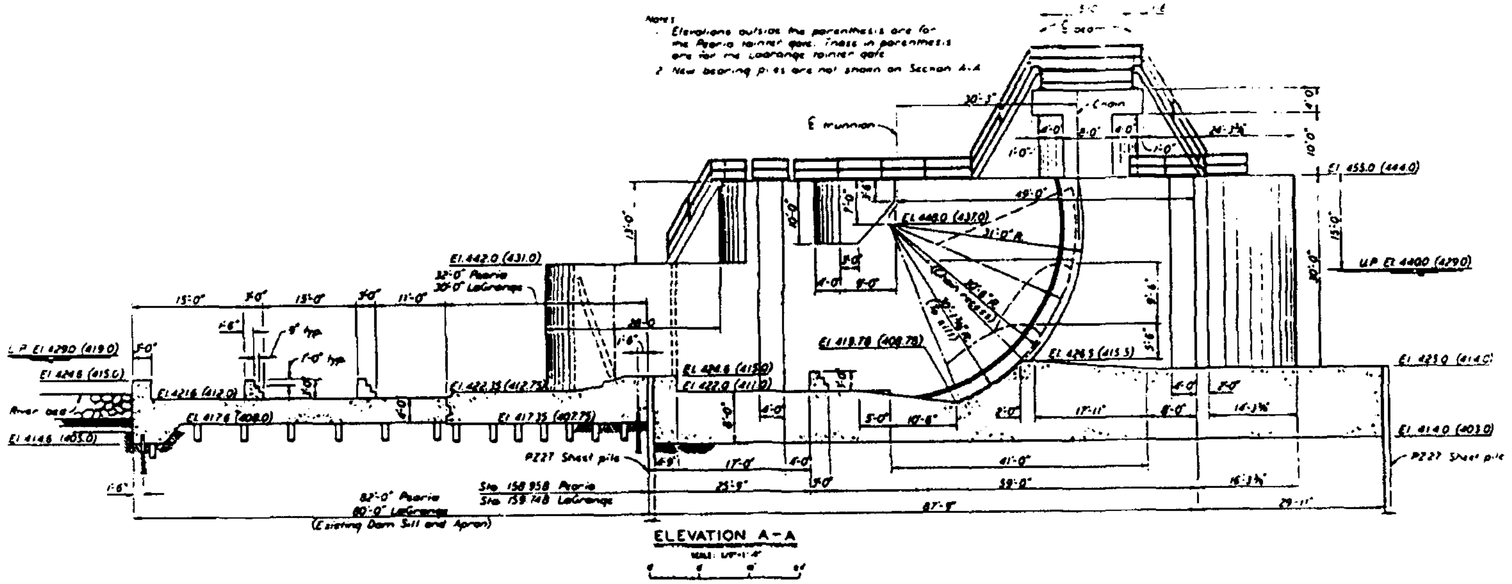

Figure 3. Peoria and LaGrange Dams— tainter gate piers, sill, and apron-elevation

Design Considerations for Dam Improvement

Ice passage and flow regulation

To alleviate some of the problems of the wicket dams, an ice passage capability and a safer and more efficient means of regulating flow had to be added to the dams. Several gate types were considered before a submersible tainter gate was selected. Other submersible tainter gates in the US Army Engineer District, Rock Island, have proven their value for passing ice and debris, and, being push-button controlled, they are a much safer and more efficient means to regulate flow than raising and lowering wickets.

The tainter gate was sized to have a flow capacity to handle all flow regulation allowing the wickets to be all raised or all lowered. In addition, before any wickets are lowered or before the last 20 wickets are raised, the tainter gate can be operated at full capacity reducing the head on the dam for easier raising or lowering of wickets.

Open-pass navigation

Since the open-pass navigation greatly reduces costs to the towing industry compared to locking through, it was essential to maintain this capability. A 1:100-scale navigation model was constructed and tested at the US Army Engineer Waterways Experiment Station (WES) to determine whether navigational safety would be preserved with proposed improvements. Initially, the District had proposed two 60-ft-wide submersible tainter gates that would have occupied 144 ft of the 540-ft navigable pass. The study indicated that this initial plan posed a potential navigation hazard. Following WES recommendations, the proposed restriction of the navigable pass was reduced to 104 ft (as was built with the 84-ft gate).

Scour protection and lock wall stability

In a second model study at WES, a 1:20-scale hydraulic movable bed model, the required riprap sizes were determined. Since one main goal of the gate was to draw ice away from the lock approach, the gate was located adjacent to the riverward lock wall where it could best meet this goal. In addition, this location allowed the operators direct access to the gate controls as opposed to the gate being located at the other end of the dam. However, locating the gate adjacent to the lock wall raised a lock wall stability concern, especially since the lock wall is founded on friction piles.

Soundings indicated that a rock-filled timber cribbing and derrick stone placement adjacent to the river wall remained intact. Just outside of this, 60-ft-long Z-piling was driven, designed to retain (by cantilever action) the existing and added scour protection adjacent to the lock wall. The piling provides insurance against rapid development of a scour hole adjacent to the lock wall which would go undetected without daily (or more frequent) soundings. Without losing navigable depth, 24 in. (0.61 m) of rock fill covered with 42 in. (1.07 m) of derrick stone were placed over the existing cribbing and derrick stone, as well as riverward of the sheet piling to 160 ft (48.8 m) from the lock wall. Since the gate was put into operation, a monitoring program has been ongoing with no scour indicated as yet.

Gate performance

Model testing for vibration. The 1:20-scale hydraulic model was also used to investigate vibration tendencies in the gate. The model could not actually monitor vibration of the gate itself, since the internal framing of the model gate did not approximate that of the prototype nor were vibration sensors attached to the model gate. Rather, load cells were attached inline to the lifting cables to monitor the magnitude and frequency of load fluctuations. The frequency of the load fluctuations was compared to the resonant frequency of the prototype lifting cables to see if the gate would have a tendency to "bounce." For certain gate positions and tailwater conditions, the model indicated low magnitude load fluctuations acting near the calculated resonant

frequency of the cables. However, the magnitude of the load fluctuations was less than the calculated trunnion friction and side seal friction (frictionless in the model). Therefore, the gate cables in the prototype would not detect the load fluctuation since the frictional forces would not be exceeded. In addition, as indicated by an hydraulic model of the similar Marseilles Dam submersible tainter gates, the vibrations are believed to be caused by the gaps between the gate and piers whereas the prototype has side seals.

Design parameters for avoiding vibration. While further research is needed to completely delineate the parameters within which submersible tainter gates can be used successfully, the following discussion is introductory guidance. It is believed that the major cause of vibration with submersible tainter gates is having simultaneous flow both over and under the gate. To this end, the district held the contractors to tight tolerances on the concrete sill and upstream gate skinplate radii to achieve a uniform 1-in. (2.54 cm) gap between the gate and sill. The sill radius was specified to be 30 ft, 1-3/8 in., ± 1/4 in., and the gate radius was 30 ft, 3/8 in., ± 3/16 in. Thus, the gap could range from 9/16 in. (1.42 cm) to 1-7/16 in. (3.65 cm). Wider gaps and flared gaps should be avoided.

It is believed that submersible tainter gates should not be used in high-head dams. Of the 82 submersible tainter gates in the Rock Island District, all operating vibration-free, none is subject to a dam head greater than 13 ft. With higher head comes higher flow in the gap between the gate and the sill. Most of the submersible tainter gates on the Ohio River that had severe vibration problems operated in dams with heads of 30 ft or more. With heads greater than 33 ft, cavitation is possible when all of the pressure head is converted to velocity head and the pressure reduces to the vapor pressure of water causing local vaporization of the water. Taller gates are also required with higher dam head, affecting the downstream skinplate shape. The Rock Island District's submersible tainter gates are submergible to 8 ft, however, the downstream skinplate is an ogee shape based upon a head on the gate crest of 12 ft.

Submersible tainter gates should be relatively rigid - an attribute more easily achieved by virtue of the dual skinplates of these gates. However, the rigidity is more difficult to maintain in longer spans. Most of the vibrating submersible tainter gates on the Ohio River had spans of 100 ft or more. The new Peoria and LaGrange 84-ft-long submersible tainter gates are the longest in the Rock Island District and most of the District's gates are 60 ft long. Helpful design guidance can be found in the narrative of report by Price (1978) which was written for computer-aided design of three-girder tainter gates. Note, however, that the Peoria and LaGrange gates are two-girder tainter gates.

Gate flow capacity. As mentioned earlier, the gate was sized to have enough capacity to have the wickets either all raised or all lowered. The hydraulic model study provided a flow rating for the gate to verify this result. While ice and debris can be passed with raised gates, flows on the Illinois River are often too low to maintain pool with the gate raised up. The submerged settings maintain pool while allowing ice and debris to pass. The model helped determine the associated discharges with various gate submergences.

Dewatering considerations

Policy reference. ER 1110-2-2901 (Headquarters, Department of the Army, 1972) sets forth the policy that, "...cofferdams on major civil works projects shall be planned, designed, and inspected by the Government in the same manner as the permanent project features." In accordance with this policy, the Rock Island District designed the cofferdam for the subject construction activity.

Crossing the wicket dam sill. The initial gate installation plan was to install the tainter gate on about the same axis as the wicket dam. This placement required removal of the sill concrete and required a cofferdam that crossed the sill (Figure 4). The wicket dam

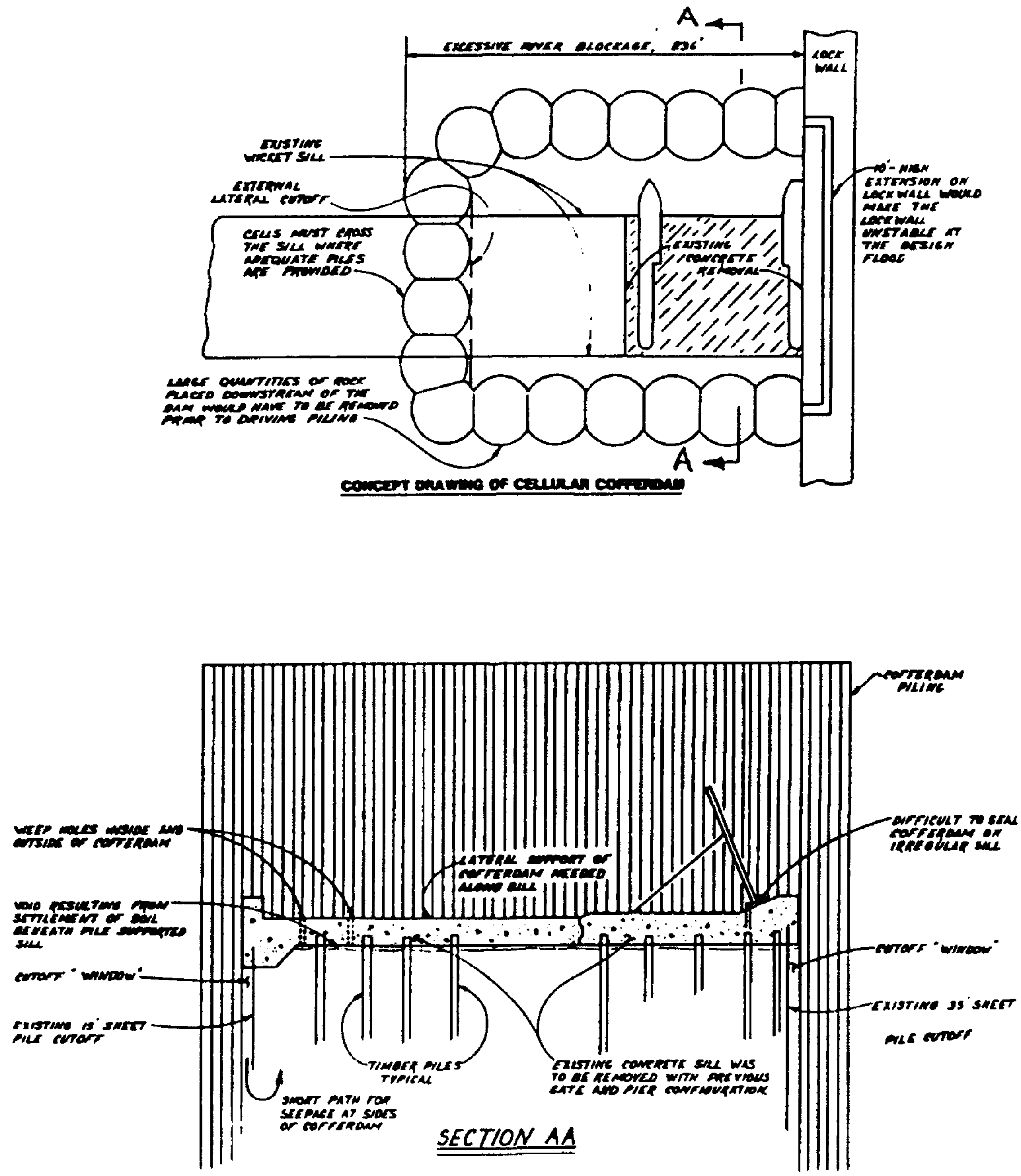

Figure 4. Cellular cooferdam-plan and section

sill has a 15-ft-long sheet-pile cutoff downstream and a 30-ft-long sheet-pile cutoff upstream. Two rows of 6-in-diam relief holes are at 10-ft centers in the apron. The concrete sill and apron are timber pile supported. During the original dam construction, several provisions were made (within the lockward cofferdam) to aid crossing the sill with the second (riverward) cofferdam: First, additional timber piles were provided to support the weight of the sand-filled cells resting on the wicket sill. Second, a lateral cutoff was provided to stop the flow of water under the sill as the cofferdam rested on top of the sill. Last, the cells of the second cofferdam crossing the sill were installed in the dry within the first cofferdam (thus facilitating the installation of a toe plate for lateral support of the piling and the sealing of the bottom of the cells).

The distance to the sheet-pile cutoff adjacent to the sill area enhanced for crossing with a cellular cofferdam is 196 ft (59.7 m) from the lock wall. Since the construction activity needed to be up to about 110 ft from the lock wall, constructing a 200-ft-wide cofferdam was considered excessive. Such a plan would also have had an adverse impact on open-pass navigation during construction. However, no closer section of the sill had such provisions for a cofferdam. It was decided that the risks would be too great to try to cross the sill with the cofferdam considering the inadequate downstream cutoff, the short flow path to the weep holes, and the added sill loading.

Single-wall sheet-pile cofferdam. The dewatering solution arrived at after much study was to construct a single-wall sheet-pile cofferdam located upstream of the wicket dam sill (see Figures 1 and 2 for location). Three sides of the cofferdam would consist of PZ-27 sheetpiling driven to an impervious layer while the fourth side consisted of the lock wall and its cutoff and a supplemental cutoff driven parallel to the lock wall. Tremie concrete was placed between the supplemental cutoff and the lock wall to form a seal. The cofferdam was supported by 10 vertical plane frames, 5 each way (2 of each 5 along the perimeter), designed with the computer program GTSTRUDL (Georgia Institute of Technology 1985). Bolted connections (A325 friction connections) were designed to allow relocation of columns, struts, and cross-bracing to accommodate various construction activities. The intersections of the plane frames were supported on H-piles. The wales were continuous with rigid connections at the corners of the cofferdam. In addition to the water load, a 30-kip concentrated load applied at any point along the upper two wales was included in the design. To protect the safety of those working inside the cofferdam, a 42-ft-diam protection cell and three protection dolphins (nine-pile cluster) were constructed upstream of the cofferdam.

While construction within the braced single-wall cofferdam was more complicated than within an open, dewatered area of a cellular cofferdam, the cost of the single-wall cofferdam was considerably less than the cellular one due to material and cofferdam-construction labor savings. The net result was a cost saving of about $1 million at each site.

Gate hoisting

A technology used heavily in conveyors for mining applications was adapted to hoisting the new gates. Round-link chain, hoisted by pocket wheels, was used with the associated motors, brakes, gear reducers, and shafting. The round-link chain is essentially maintenance-free being resistant to wear (especially at the low operating speeds of hoisting tainter gates) and corrosion. To hoist the 126-ton gates with the required factor of safety, two round-link chains were required on each side of each gate. The chain used is 34 mm x 126 mm links manufactured from an alloy equal to AISI 8620 steel to DIN Standard 22252. The chains are prestretched to assure uniformity of length. Prestretching plus turnbuckle adjustment on the gate assure equal load among all chains.

Construction Experience

Cofferdam construction

Both contractors assembled the cofferdam framing above the construction area and then lowered the 580 tons of framing into the sheet-piling by controlling the load with four hydraulic jacks at Peoria and nine hydraulic jacks at LaGrange. Once the cofferdams were pumped down, a seepage problem became apparent at both sites. The pressure of the water from the upper pool acting between the lock wall cutoff and the parallel supplemental cutoff caused the supplemental cutoff piling to separate from the tremie concrete seal. By postconstruction assessment, the supplemental cutoff should have had steel studs embedded in the tremie concrete (although this may not have been a complete solution). However, another means had to be developed once the problem was discovered during construction.

The solution was three-tiered: First, a concrete cap was placed over the supplemental sheet-pile cutoff and anchored to the lock wall; second, additional piling was driven upstream and connected to the cofferdam and capped with concrete to lengthen the seepage path; and third, voids that had developed in the seepage path under the lock wall were filled with sand and sodium silicate grout. This was another lesson in the importance of design details.

General construction

The Peoria contractor elected to ship the gate in three segments, the two end shields separate from the main barrel of the gate. Whereas the LaGrange contractor's gate was completely shop fabricated, the Peoria gate had a fabrication error in that the radius at the top of the upstream skinplate was greater than the radius at the bottom. Thus, the gate deviated from the desired uniform gap between the gate and the sill. However, the maximum gap did not exceed 2 in. The LaGrange contractor had a different problem with the tainter gate. The spud barge upon which a stiff-leg boom was holding the gate drifted while two cranes on land were guiding the gate to be set on timbers. The resulting lateral load on the boom buckled it, and the gate was dropped in the river. No significant damage was done, however.

Conclusion

The modification of Peoria and LaGrange wicket dams by addition of a submersible tainter gate has greatly improved operational safety, effective flow regulation, and passage of ice and debris. Submersible tainter gates should be considered for similar applications as they can be operated successfully; certain design considerations were identified that should signal additional study for some applications. The reader is also encouraged to sift through design alternatives to find the most effective and economical solution in these times of shrinking budgets. Such was the case in developing the single-wall cofferdam design. Finally, we will do well to remember that details are often very important to the final outcome.

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Finite Element Analysis of Miter Gate Anchorage

by

Bruce C. Riley, PE[1]

Abstract

This paper describes a finite element analysis which was used to determine the suitability of using the existing US Army Engineer District, Pittsburgh, miter gate anchorages on two new lock projects currently under construction on the Monongahela River. Although not overly complicated, the analysis is a good example of using advanced analysis techniques to solve some of the unique problems encountered by engineers within the Corps. This analysis used the Intergraph Rand-Micas finite element analysis package, incorporating their solid brick elements to model some typical eyebar anchorages.

This paper was presented by the author at the Engineering/CADD Symposium III in Kansas City in July 1990, in the session for structural engineering applications.

This paper describes a finite element analysis for the miter gate anchorages for two new locks which are currently under construction on the Monongahela River in the US Army Engineer District, Pittsburgh. This work started out as a new design, but ended up as an analysis of a design which has been used on several other projects in the Pittsburgh District. The Operations and Maintenance people resisted changes to the anchorage, and properly so, since spare parts already in inventory would not be useful and new spares would have to be procured for these two projects. In today's climate of standard designs for everything from guide walls to miter gates and machinery, it really did not make sense to change the anchorage. We, therefore, used the same design and checked it for the highest loads at the two projects. In the new engineering manual on miter gate design, the anchorage system that we use in Pittsburgh is referred to as "the alternate system."

Figure 1 is taken from the contract drawings for our Point Marion Lock and Dam and shows the plan view of the miter gate and anchorage. The anchorage consists of two triangular embedded steel frames, one perpendicular to the face of the lockwall and one 10 deg off parallel to the chamber face. The load from the miter gate is transferred to these frames through the gudgeon pin by two anchor bars, the long bar connecting directly from the gudgeon pin to the embedded frame, and the short bar connected by the link pin to two gudgeon plates which sandwich the long bar at the gudgeon pin. This allows both bars to connect to the gate and provides some flexibility in the connection. The anchor bars are connected to the frames through wedges which provide the adjustment for plumbing the miter gates. Figure 2, an elevation view of the anchorage, is taken through the short bar and gudgeon links. The anchor bar described in this analysis however is the long

[1] US Army Engineer District, Pittsburgh; Pittsburgh, PA.

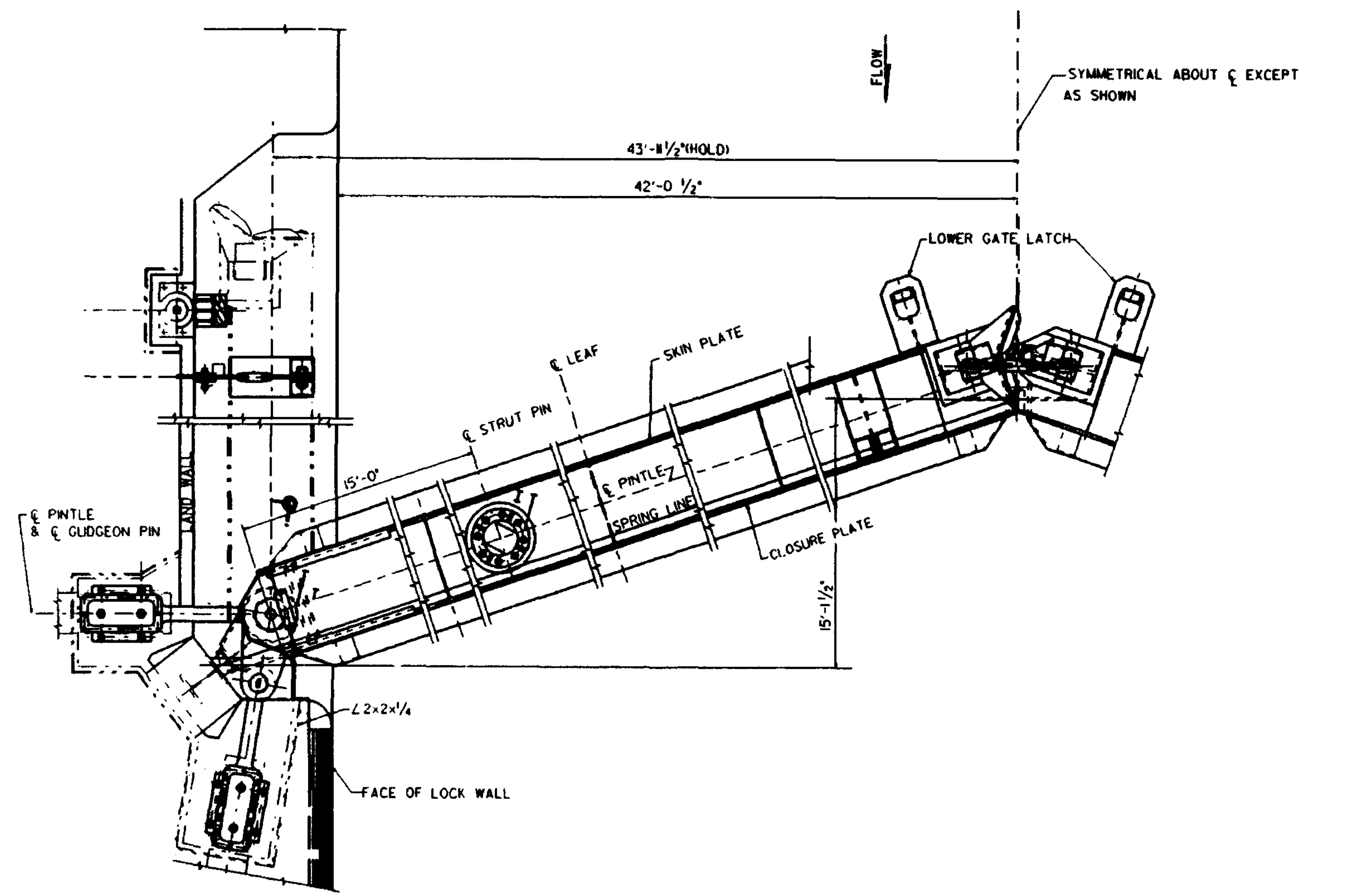
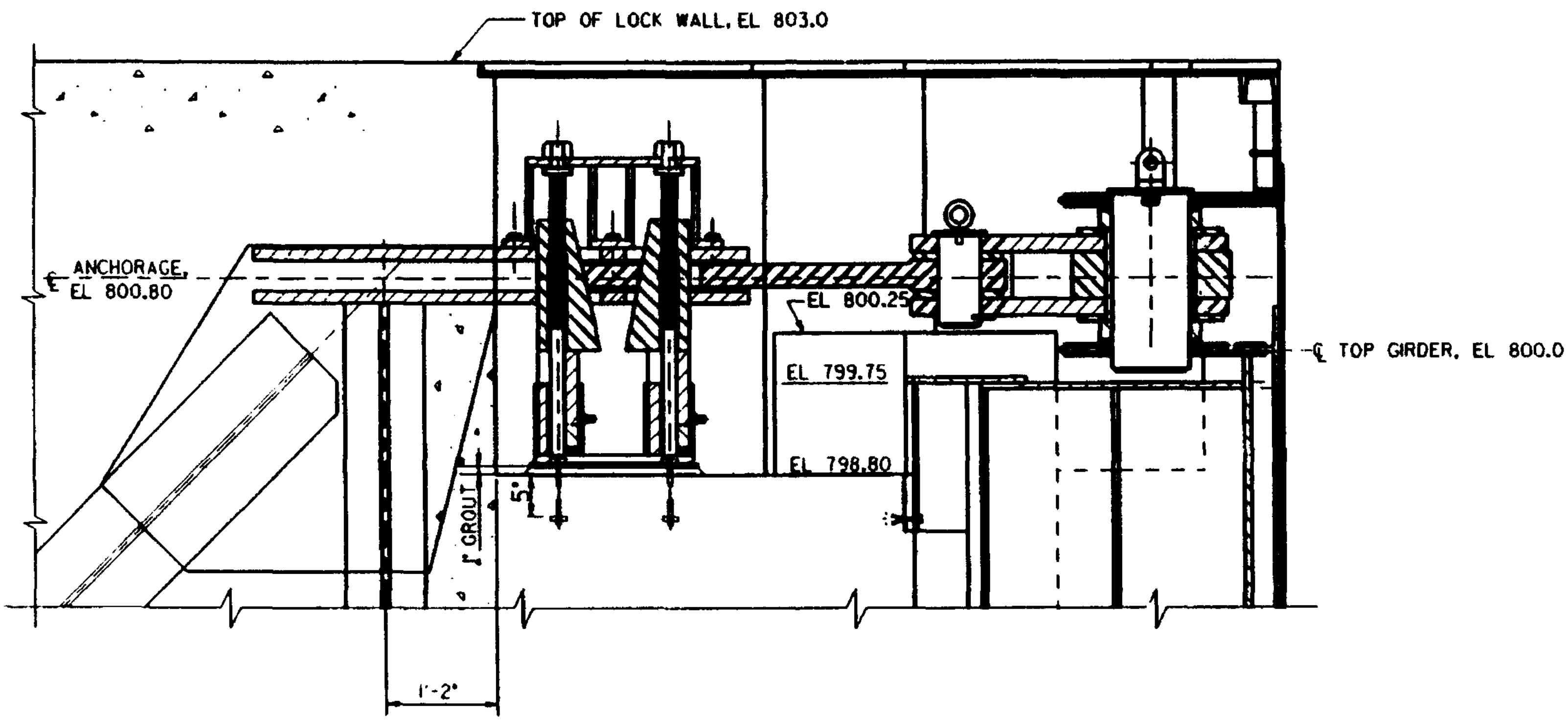


Figure 1. Plan view of gate and anchorage



Figue 2. Elevation view of anchorage

anchor bar. Figure 3 shows a detail of the gudgeon links. Figure 4 shows the details of the anchor bar. It is approximately 5-1/2 ft long. The stem is 3 in. thick and the ring is 6 in. thick. This detail comes from the contract drawings, but there is a difference in the one used for the model later, because the bar described in this analysis has a 12-in.-diam hole for the gudgeon pin while the one which we eventually used has a smaller diameter hole for a 10-in.-diam pin. I am going to describe only the analysis of the longer anchor bar, but I also analyzed the short bar, the gudgeon links, and the embedded anchorage using IRM.

I used the Thin Shell model from the structural graphics interface and the 8-node brick element which IRM calls the "SOL" element (Figure 5). This is a 3-dimensional model, and I had to draw the anchor bar, place the nodes, and then generate the elements.

The supports chosen for the anchor bar were conservative, while still providing for a realistic model. The only nodes which are used to resist the translation in the X-direction are the two directly behind the wedges, when in fact the two nodes on either side of these are also restricted from X-direction translation. By concentrating the reaction at a point, I can check a condition which is more conservative and which, due to fit and/or machining inaccuracies, could exist. To completely stabilize the anchor bar model, all the nodes were fixed around the wedges in both the Y and Z directions and around the inner circle of the eye in the Z direction. This keeps it in the X-Y plane, which is true of the actual anchorage system, too. All nodes are fixed against rotation, since the model has 6 deg of freedom, but the element has only 3.

The loads for the anchor bar were calculated using EM 1110-2-2703, "Lock Gates and Operating Equipment," and EM 1110-2-2602, "Planning and Design of Navigation Lock Walls and Appurtenances," (Headquarters, Department of the Army, 1984 and 1960, respectively). The loads represent the highest loading on the bar and are the reactions to the gate leaf dead weight and operating strut force, increased by 10 percent for impact. The axial load on this anchorage member is 174.2 k. Using a friction factor of 0.2 for the bronze bushing, which is a typical value for bronze on steel, the axial load causes a frictional force of 34.8 k. This frictional force should be distributed around at least one-half of the inside face of the circular section. I tried using a surface traction load but could not get the expected results. The inside surface area is 254.5 sq in., and I used a traction force of 0.137 ksi, for a total force of 34.9 k. This did not work properly, so I used couples to roughly model the frictional moment. I feel this is satisfactory but causes a concentration that may be more severe than the actual case. I used eight nodes on the Y-axis and eight nodes on the X-axis. This amounted to two couples on four different planes which resulted in the distribution of one-half of the frictional moment to the center elements and one-fourth to the upper and lower elements. The computed moment is 34.8k $\times$ 6.75 in. or 235 k-in. The couples result in $2 \times (3.28125 + 3.28125 + 1.09375 + 1.09375) \times 13.5$ in. = 236 k-in.

Now that the model is built, the supports defined, and the loads are applied, we need only analyze it. This is where we let the program do the real work. After the analysis, we can look at some of the results displayed both graphically and through text files. Figure 6 shows the deformed shape of the bar under the combined axial load and frictional moment, if it were free to assume this shape. This is exaggerated 100 times. The actual anchor bar is restricted from moving entirely through these deformations by the short anchor bar/gudgeon link connection.

Using the IRM post processor, we have the ability to view stress contour plots. A plot of σ_x, or the normal stress on the plane perpendicular to the X-axis, for the pure bending case only, produces a pleasing representation since the contours display stresses just as we would expect to see them. They are smooth and evenly spaced, showing tension on top and compression on the bottom, and the neutral axis is highly visible along the center of the bar. A contour plot of the same stress distribution, σ_x, except using only the axial

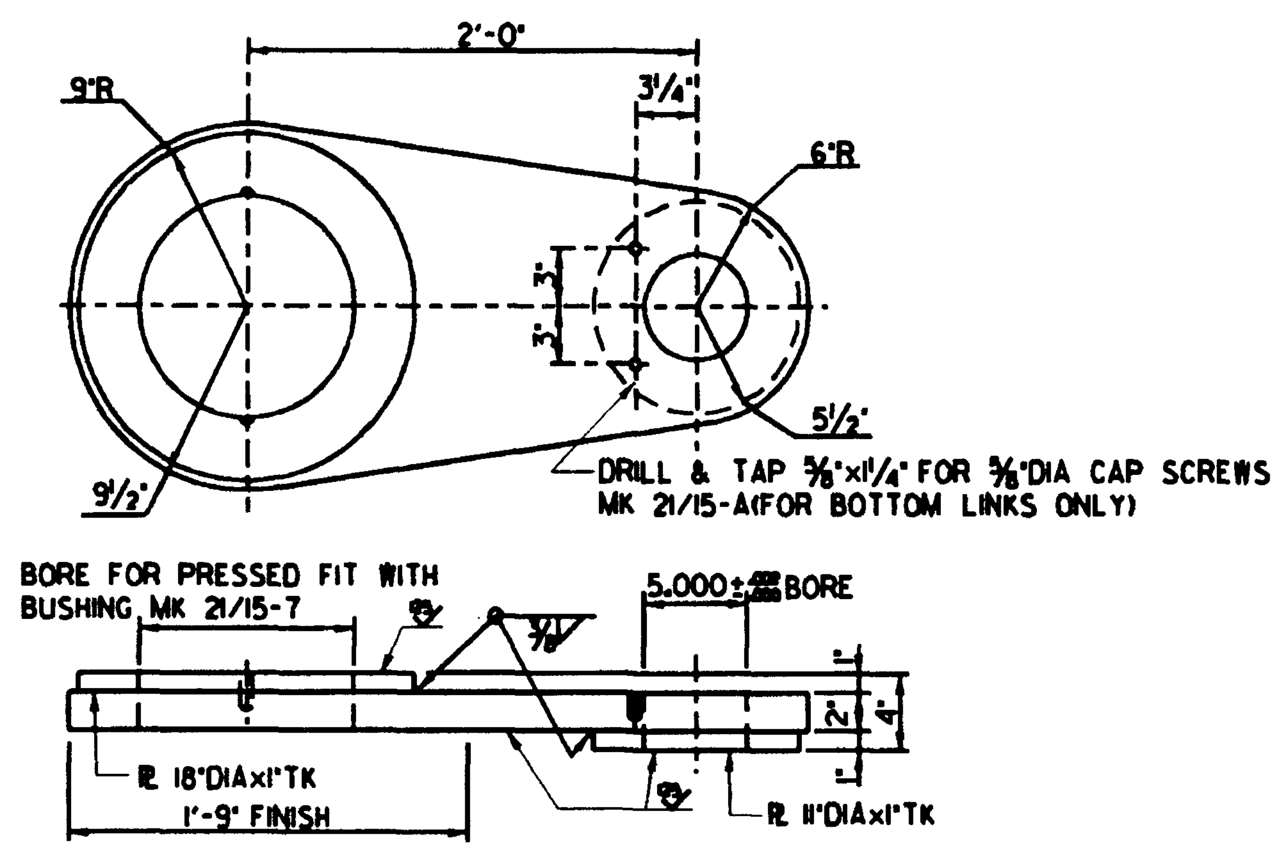


Figure 3. Gudgeon link

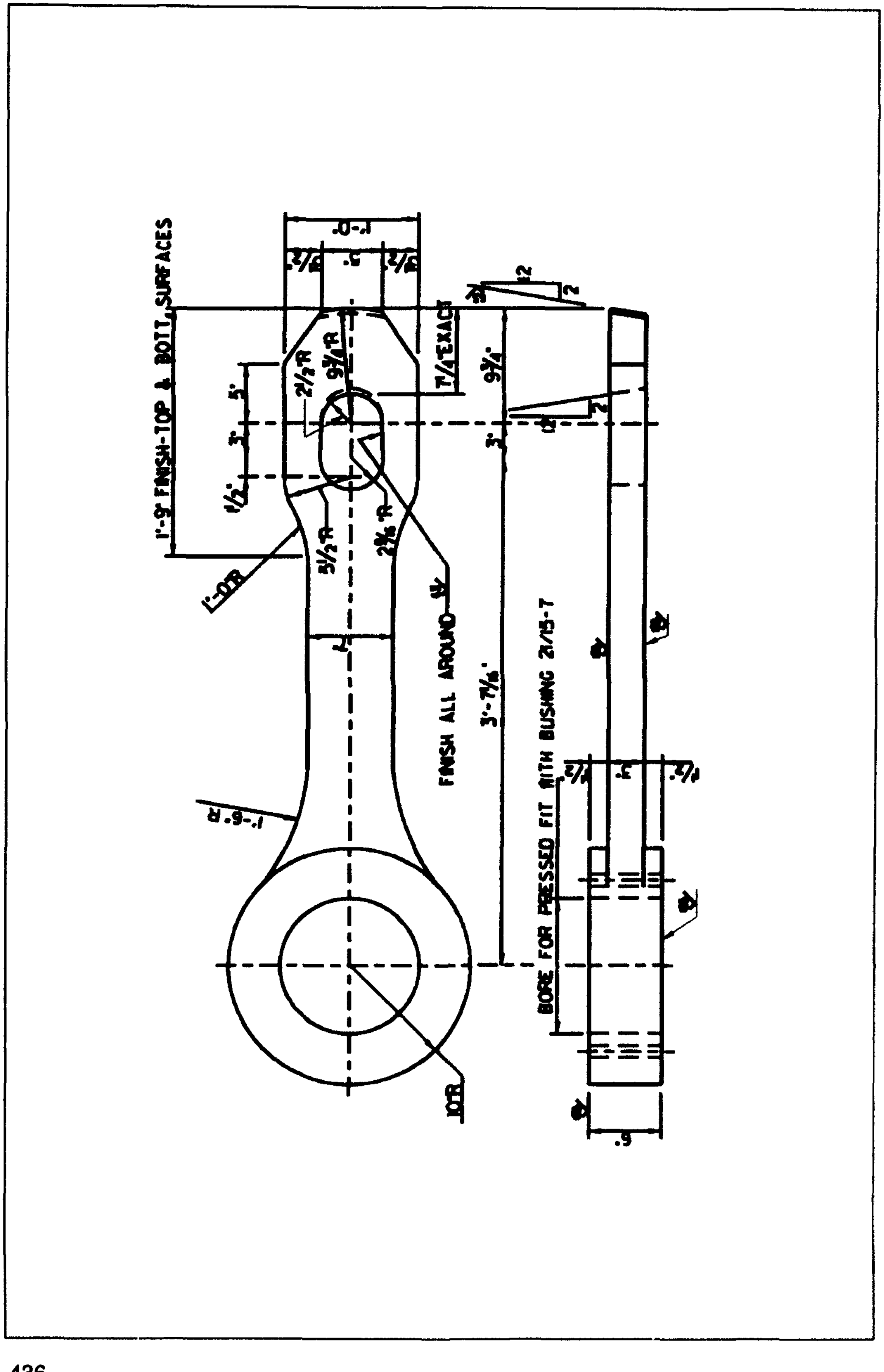


Figure 4. Anchor bar

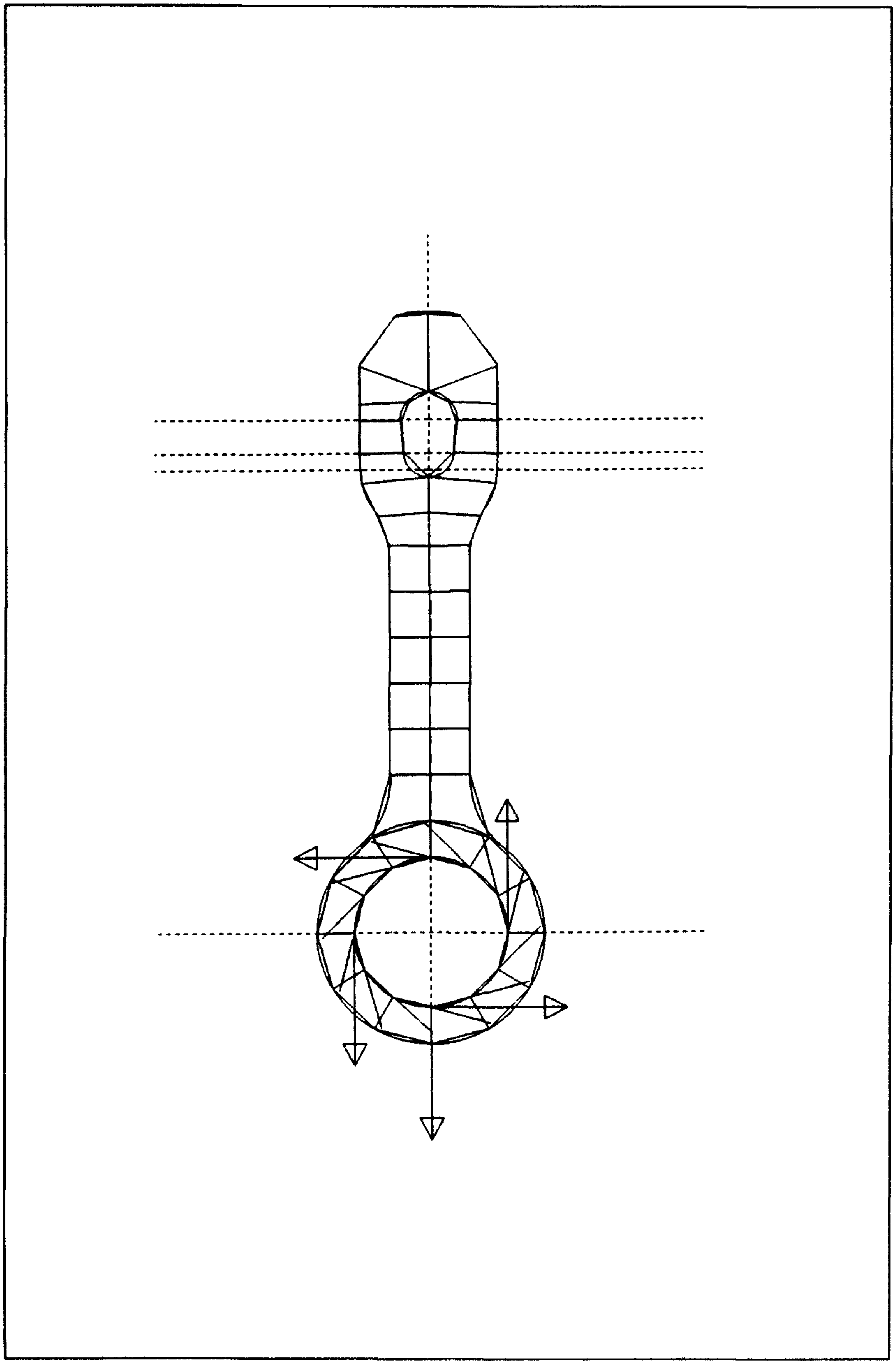


Figure 5. Anchor bar model

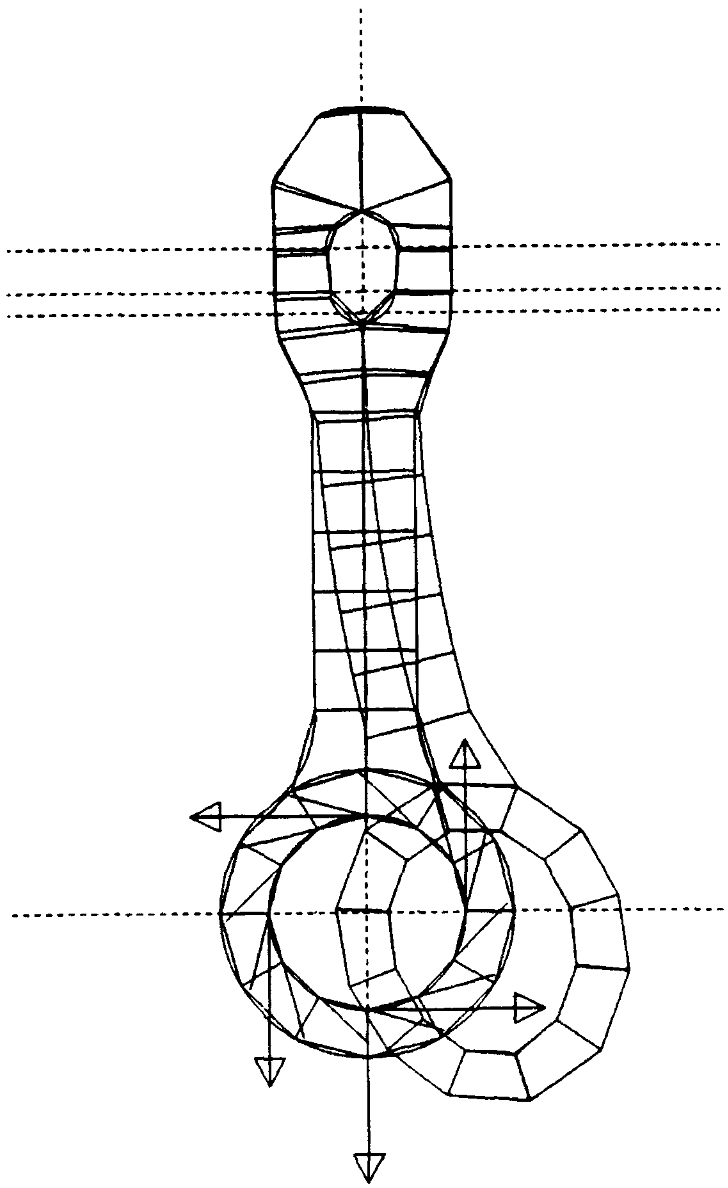


Figure 6. Deflected shape

load, shows contours which are not as nicely distributed as the pure bending case, and may, in fact, be the result of some quirk in the model such as a node slightly out of alignment. I would expect to see a more constant stress distribution along the slender portion of the bar. A plot of the normal stress distribution caused by the combined effects of both the axial force and the frictional moment is as would be expected. The neutral axis is moved down, very close to the bottom of the bar, due to the high tensile stress of the axial load as it overcomes the relatively smaller compressive stress of the bending moment.

Once the analysis is complete, we need to go to the post processor through the alphanumeric interface. Here we can sort through and select the output that we want to view or print out. I usually go first to the MinMax reports to get a feel for the range of stress and displacement that the structure experienced. For the anchor bar, the maximum element normal stress is 13.2 ksi tension and 4.8 ksi in compression. These stresses occur in elements 65 and 92, respectively. We see also that these stresses are caused by different load case or combinations. The maximum shear is 5.5 ksi and is found in the elements which are closest to the point of application of the axial force. The maximum nodal displacement is 0.05 in. for the combined load case. One thing that becomes apparent in reviewing this analysis is that a finer mesh could have been used to refine the analyses and the results. The maximum element stress reported is 13.2 ksi, but this is the stress at the center of the element. The actual maximum is at the extreme fiber and is closer to 17.9 ksi. The program averages the stress over the face, and, in this case, it averages the maximum at the extreme fiber and the minimum or zero at the neutral axis.

When I first began looking at this anchorage, I was not thinking about the frictional moment and modeled only one-half of the bar. Any time there is symmetry in a problem we should take advantage of it. For this anchor bar under axial load only, we need only look at one-half of it. The support conditions are the same as the whole bar except that the nodes would be fixed against y axis translation along the line of symmetry. Of course, in this case the frictional moment destroyed the symmetry, and the full bar had to be modeled. It is interesting, none the less, to look at the half-bar model. The contour plots are similar to those of the full model. I have tabulated the normal stress σ_x for a number of elements from both of the models and the differences are very slight, on the order of 0.01 in., as are the differences in the displacements, on the order of 0.001 in. Unfortunately, there are not too many problems where this can be done, but when possible, the symmetry should be used to our advantage. It can reduce the amount of work required and the amount of computer time necessary to analyze a problem.

Structural Damages from Hurricane Hugo

by
Rick Lambert, PE[1]

Abstract

The following presentation is an account of structural damages caused by Hurricane Hugo, which made landfall in the vicinity of Charleston, SC, on the evening of 21 September 1989.

Hurricane Hugo was the most powerful hurricane to strike the east coast in the past century. This was a category 4 storm with sustained winds of 135 mph and gusts reported as high as 175 mph.

Hurricane Hugo caused many structural failures and much damage to buildings and other structures in the coastal area of South Carolina. Much can be learned about wind forces by observing the structures which were destroyed or damaged by this powerful hurricane.

Path of Hurricane Hugo

Hurricane Hugo was the most powerful hurricane to strike the east coast in the past century. Hugo tracked across the Atlantic and through the Caribbean from 9-18 September 1989, leaving a path of destruction in the Virgin Islands and Puerto Rico. The storm's impact with the island land masses reduced wind velocities from 140 to 105 mph and caused the storm to veer in a more northwesterly direction as it headed for the southeast coast of the United States.

As the storm neared the southeast coast on the afternoon of September 21, it increased in strength from a Category 2 storm with winds of 110 mph to a category 4 storm with 135 mph winds. The storm's forward speed also increased to 20 mph, which reduced the time that people in the Charleston area would have for last minute preparations and evacuations. The storm made landfall in the vicinity of Charleston, SC, at midnight on Thursday, September 21, 1989.

Hurricane Hugo struck the South Carolina coast and its effects were felt throughout the entire coastal area. Sustained wind speeds were estimated to be 135 mph in the vicinity of Charleston with gusts reaching 175 mph. The storm continued inland with the eye passing between the cities of Columbia and Sumter and on to the North Carolina border. Hurricane force winds were reported in the city of Charlotte, NC, some 200 miles inland from the storm's original impact with land.

Estimated Dollar Damage

As a result of Hurricane Hugo, 24 of South Carolina's 46 counties were declared disaster areas. Total economic damages were estimated to be in excess of $5.8 billion. More than 4.5 million acres of forest, representing 36 percent of the state's woodlands were

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severely damaged. More than 6.7 billion board feet of timber valued in excess of $1 billion were damaged and only about 25% of this wood could be salvaged. Damages experienced equated to $1,686 for every citizen of the state.

The eye of a hurricane is the calm area near the center of the storm around which most of the destructive winds are located. Hurricane Hugo's eye was about 25 miles in diameter and it passed directly over the city of Charleston. The western wall of the eye was located near Seabrook Island and the eastern wall limit was located just north of the Isle of Palms.

Charleston and the state of South Carolina were in the direct path of Hurricane Hugo. The approximate limits of the storm's eye and its direction are indicated by the red arrows. Although the eye of the storm passed directly through Charleston, the city should consider itself fortunate. A storm's most destructive quadrant is the northwest quadrant which experiences the most severe tidal surge. This portion of the storm struck the sparsely populated area in the vicinity of the Francis Marion National Forest. Had the eye of the hurricane made landfall a short distance south of Charleston, the damages experienced by the city and the potential for loss of life would have been much greater.

Tidal Surge Struck During High Tide

One of the most feared parts of a hurricane is the accompanying tidal surge. The tidal surge is generally located in the northwest quadrant of the storm and results from winds pushing a wave of water inland. The surge is very dangerous and results in severe damage and threat to life. Hurricane Hugo struck the South Carolina coast near the normal high tide. The resulting tidal surge was about 17 1/2 ft (elev 19.97 msl) above the normal tide elevation in the Cape Romain area just north of Charleston. This is equivalent to a wall of water about the height of a two-story building.

The newspaper headline the morning after the storm read "Charleston is Ground Zero", which accurately described how the area looked after the hurricane struck.

Wind and Water Destruction

Many of the state's coastal structures were totally destroyed from a combination of hurricane wind forces and tidal surge. It was difficult to assess the failure mechanism of many of the totally destroyed homes because in many cases, there was little left to observe. Some houses seemed to have vanished and only a pile of debris remained at the site of others.

Beach houses with unreinforced or under-reinforced masonry piers and shallow foundations near the front beach were severely damaged or destroyed. Piers on one house shifted and racked back from wind and tidal surge forces. The masonry piers on a relatively new beach house failed and the house fell to the ground. In many cases, houses which were not securely anchored down floated away.

Structural Damage

Roofing

All areas in Hugo's path experienced severe damage. Extensive roof damage was inflicted to many buildings in the historic downtown area of Charleston. All forms of roofing were damaged by wind: asphalt shingle, metal, and various forms of single ply roofing membranes all experienced roofing failures.

Many buildings lost large sections of the roof structure. In some cases the entire roof framing was pulled off by suction forces. Interior finishes and contents of many buildings were damaged from roof failures and accompanying leaks. Roofing on many church steeples was battered. Steeples and entry canopies toppled on several churches.

Cladding and brick veneer

Cladding was also pulled from many buildings by wind. Brick veneer pulled off buildings. Brick veneer was pulled off the upper portion of a tower at the municipal auditorium, fell on the roof below, and damaged the roof framing.

Unreinforced masonry

Unreinforced masonry used on the exterior walls failed from wind loading. Unreinforced masonry used on front beach was no match against the tidal surge which occurred during the hurricane. Hurricane force winds caused several old unreinforced masonry buildings such as this one in the downtown area to collapse in a pile of rubble.

Metal frames

Many metal buildings were damaged. End walls on a metal warehouse building failed. A metal building frame which formed an entry canopy was relocated to the rear of the building, in the swimming pool. A new gas station was extensively damaged by hurricane wind forces. Many mobile homes were also severely damaged.

Trees and tall slender structures

Tree damage was extensive throughout the state. Especially hard hit was the The Francis Marion National Forest, north of Charleston. Several hundred square miles of the state's timberland were devastated. Much damage to residential structures resulted from trees falling on them. Many of South

Carolina's residents found this type of damage as they surveyed their homes the morning after the storm. Pine trees snapped like matchsticks and fell on roofs. Others were uprooted and struck buildings. Tree impact was probably responsible for at least one-half of the structural damages which occurred.

Trees were not the only tall slender structures to be destroyed by wind. Many power poles were knocked down by the wind. Radio and television towers were knocked down by hurricane wind forces. The District's 450-ft radio tower fell, as did the 1,600-ft-tall Channel 24 television tower. Tall lighting poles on a stadium fell and impacted the concrete structure causing extensive damage.

A concrete pile cap, which was the foundation of a large light pole, was not anchored down. The pile cap lifted off of the timber piles and the light pole fell to the ground. Precast concrete stadium slabs at a local high school were lifted by hurricane wind forces. These slabs are 3 in. thick and weigh 49 psf on the horizontal projection. Hurricane wind forces toppled a large container crane at the port of Charleston. Hurricane Hugo damaged a swing span bridge, causing it to fall into the Intracoastal Waterway.

Marinas

The storm surge also caused damages to structures other than buildings. Marinas were rearranged by the storm. Many boats were moved from their moorings to high ground. A new marina was constructed by Hurricane Hugo, locally referred to as the Goat Island Yacht Club. Boats which were all securely moored at the Wild Dunes Yacht Harbor were relocated to the opposite bank of the Atlantic Intracoastal Waterway during Hurricane Hugo.

Cars and bridges

Boats are not the only vehicles that float. A car came to rest on a concrete table after floodwater receded.

Flooding washed out the abutments to many bridges and culverts. The protective dunes on the front beach were destroyed and had to be rebuilt soon after the hurricane to protect against tidal flooding from seasonal high tides.

Conclusion

In summary, much can be learned from studying the types of structural damages

which typically occurred as a result of the most powerful hurricane to strike the east coast in the past century. Roofing and cladding damages occurred to nearly every building in the hurricane's path. The importance of proper connection detailing, particularly for roofing and cladding, cannot be overstressed. Also, anchorage of roof framing members for wind forces and foundation anchorage in flood prone areas is of utmost importance. Trees pose a hazard to buildings, and consideration should be given to maintaining clear zones particularly around essential facilities.

It has been nearly 2 years since the hurricane struck. The debris was cleared, repairs were made, and new buildings were built. The city of Charleston has endured wars, earthquakes, and hurricanes in the past. Charleston endured Hurricane Hugo, but the storm will not be soon forgotten.

Structural Aspects of Caldwell Trucking Well No. 7 Superfund Site Design

by

William F. Strobach, PE[1]

Abstract

The Caldwell Trucking Company site is located in Fairfield Township, NJ. The Caldwell Trucking Company operated a septic waste hauling, treatment, and disposal facility that had used unlined lagoons at this site for 25 years. These operations have caused contamination in the soil and groundwater which disrupted the use of potable water from Well No. 7 since 1981. This paper is a case study of the development of the water treatment system, consisting of air stripping and carbon absorption. This system was used as the remedial action for the Well No. 7 contamination.

This project is the first in-house design of a superfund project (requiring structural design) undertaken by the US Army Engineer District, Kansas City.

The structural components of the water treatment system consist of an air stripper tank, a clearwell structure housed inside of a masonry building, two carbon unit tanks, an airstack, and the related foundation structures.

Design loading criteria, structural considerations, and design procedures for each component are discussed. The input of a structural engineer on the design team is required at an early stage in the development of the project as structural aspects in this case changed the layout of some of the components in this water treatment system.

Overview

Superfund program (general)

The Corps of Engineers' involvement is undertaken by the US Army Engineer District, Kansas City, which is responsible for 23 states, and the US Army Engineer District, Omaha, which is responsible for 27 states. The division of the states to the districts is based on EPA regional boundaries. For instance, the Kansas City District is responsible for the state of Nebraska, because Nebraska is situated in EPA Region VII, one of the EPA regions assigned to Kansas City District. EPA has identified approximately 1,200 contaminated sites in the United States. New Jersey has the most contaminated sites of any state, approximately 110. At the request of EPA, the Corps of Engineers gets involved in approximately 20 percent of the contaminated sites. The remainder of the sites are handled by the EPA, state agencies, or the primary responsible party (for the contamination).

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Caldwell Well No. 7

The subject project is located in the town of Fairfield, NJ, approximately 20 miles west of New York City. An architect-engineering firm initiated design of the subject project and proceeded to 35 percent of a design, at which time the contract was terminated. The Kansas City District then undertook this project as its first in-house design effort and proceeded to final design. The Caldwell Trucking Company site is a 12-acre site located in a light industrial area of Fairfield Township, Essex County, New Jersey. The Caldwell Trucking Company operated a septic waste hauling, treatment, and disposal facility and had utilized unlined lagoons at this site for 25 years. These operations appear to have caused organics and heavy metals contamination in the soil and groundwater at this site. The extent of the contamination is shown in Table 1.

Table 1
Maximum Expected Influent and Required Cleanup Levels

Contaminant	Maximum Expected Influent ppb	Federal MCL's ppb	New Jersey MCL's ppb
1,1,1 Trichloroethane	1,400	200	26
Trichloroethene (TCE)	2,000	5	1
Tetrachloroethene	1,200	5	1
Chloroform	19	100	100
Carbon Tetrachloride	54	5	2
1,2 Dichloroethene	53	7	10
1,1 Dichloroethane	13	[1]	[1]
1,1 Dichloroethene	90	7	2
1,2 Dichloroethane	48	5	2

[1] No criteria set.

As Table 1 illustrates, several of the contaminants at the Caldwell Well No. 7 site far exceed the US and New Jersey standards. For instance, one of the contaminants, trichloroethene (TCE) is 400 times the US standard and 2,000 times the New Jersey standard. TCE is a colorless liquid used as an industrial solvent and degreaser for metal components. Consumption by humans can cause damage to the liver. It is also classified as a B2 carcinogen, a probable cancer-causing agent, and increased cancer risk occurs in direct proportion to the concentration in the drinking water.

Remedial action planned

To reduce the contaminant concentrations in Well No. 7 and to bring its drinking water to acceptable health requirement standards so that the lost potable water may be restored back into service, a water treatment system will be utilized. In looking at structural aspects and total design quality, the primary objective was to use a system with the lowest possible life cycle cost, while still attaining all environmental and health requirements. It is anticipated that Well No. 7 treatment system will be operating for an indefinite period of time, should be designed to minimize operation and maintenance costs, and is to be constructed of materials that will ensure longevity. The proposed water treatment system will be located adjacent to the existing Well No. 7 building as shown in Figure 1. The proposed water treatment system consists of the water from Well No. 7 being pumped to the top of a packed air stripper tower which is then circulated through the tower to remove the contaminants in the water. The "stripped air" is then heated and circulated to two granular activated carbon filter units with an exhaust stack to vent the air used in the contaminant removal process.

Design Loadings

General criteria

There are no well-defined criteria for use on design of superfund projects. For design loadings, a combination of military criteria and civil works criteria was used.

Importance factors

The first step in considering structural aspects of a project is to determine its importance factors. There are three criteria that address occupancy importance factors. These criteria are TM 5-809-1, (Headquarters, Departments

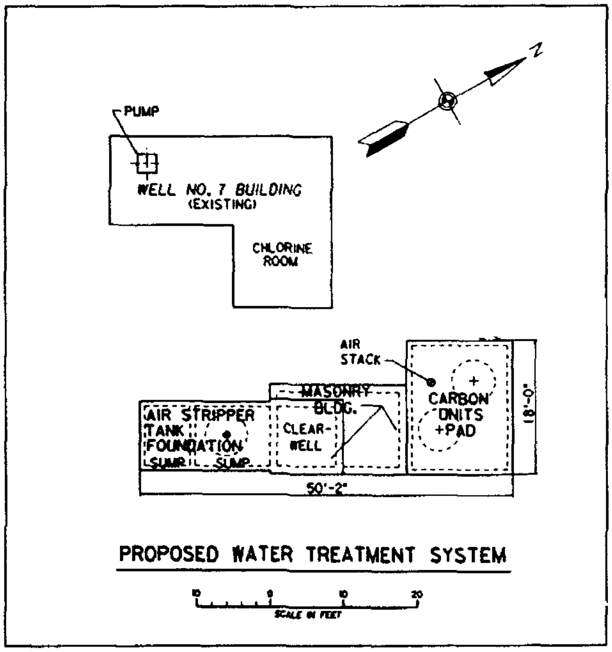


Figure 1. Plan of proposed treatment and existing Well No. 7

of the Army and the Air Force 1986), American National Standards Institute (ANSI) A58.1-1982 (1982), and TM 5-809-10 (Headquarters, Departments of the Army, the Navy, and the Air Force, 1982). Table 1-1 of TM 5-809-1 assigns three different categories for occupancy degree of risks involved but does not list a facility that is compatible to this project. Table 1 of ANSI A58.1-1982 lists utilities required in an emergency as Category III and other structures except those listed in this paragraph as Category I. Paragraph 3-5 of TM 5-809-10 lists facilities that best address this project. Utilities required as emergency facilities, paragraph 3-5.a.(3), are listed as an essential facility (I = 1.5). Central utilities such as water that are not covered in paragraph 3-5.a.(3) but that serve large areas are listed as high risk (I = 1.25). All structures not covered by essential or high-risk categories are given I = 1.0. In looking at this particular project, Well No. 7 has not operated since 1981, and the town of Fairfield has six other wells that are currently functioning. Well No. 7 is intended to be used only for peak demand. This well does not service a particularly large area in that Fairfield only has a population of 7,000. Based on these considerations, an I = 1.0 was assigned. With an occupancy importance factor of I = 1.0 (Category I), the wind-load importance factor can now be determined. Table 5 of ANSI A58.1-1982 (ANSI 1982) lists windload importance factors, and, for Category I, lists an I = 1.00 if project is more than 100 miles from hurricane oceanline and I = 1.05 if project is at hurricane oceanline. A windload importance factor of I = 1.05 was assigned due to the project's proximity to the hurricane oceanline.

Wind velocity pressure coefficients

The A/E who previously worked on the 35-percent design of this project used an exposure B category. In looking at paragraph 6.5.3 of ANSI A58.1-1982 (ANSI 1982), use of this exposure category shall be limited to those areas for which the terrain representative of Exposure B (suburban areas or terrain with numerous closely spaced obstructions having the size of single family dwellings or larger) prevails in the upwind direction for a distance of at least 1,500 ft or 10 times the height of the structure, whichever is larger. Upon talking to individuals who visited the site and looking at photographs, although there are single family dwellings and commercial buildings nearby, the site is located in a clearing and these obstructions are located more than 1,500 ft away. Therefore, an exposure C category should be used. Using an exposure C category results in a 57-percent increase in wind load for this project when the appropriate factors are substituted in the wind-force equations. Combining equations (3) and Table 4 (other structures) of ANSI A58.1-1982 (ANSI 1982) results in $F = 0.00256\, K_z\, (IV)^2\, G_z\, C_f\, A_f$ (Equation 1). In comparing exposures B and C, the only two variables from Equation 1 are K_z and G_z. Values of K_z and G_z from Table 6 and Table 8 of ANSI A58.1-1982 (ANSI 1982) for a height of 40 ft for Exposure B are 0.57 and 1.46 and for Exposure C are 1.06 and 1.23. Multiplying these factors for Exposure B gives 0.57 × 1.46 = 0.832 and, for Exposure C gives 1.06 × 1.23 = 1.304 or a 57-percent increase in load.

Wind speed

The project site is located nearest Picatinny Arsenal, and from Appendix A of TM 5-809-1 (Headquarters, Departments of the Army and the Air Force, 1986), the wind speed to be used at this location for a 50-year mean recurrence interval is 75 mph.

Seismic zone

The project site is located near Picatinny Arsenal, and from Table 3-5 of TM 5-809-10 (Headquarters, Departments of the Army, Navy, and Air Force, 1982), this location is designated as Seismic Zone 2.

Soil loading

The foundation walls are considered to be fixed or hinged at the support. Therefore, wall movement under loading will be relatively small or nonexistent and at-rest soil loadings will develop according to paragraph 3-4.c. of EM 1110-2-2502 (Headquarters, Department of the Army, 1989). An at-rest submerged soil loading will be used as the maximum design loading condition.

Design Considerations—Air Stripper Tank

Supporting slab design

Upon joining the design team, a sanitary engineer approached me concerning the supporting slab for the air stripper tank shown in Figure 2. He said he needed a 5-ft-diam opening in the slab with the tank supported on a peripheral ring. However, upon looking at this supporting slab closer, this configuration was a structural engineer's nightmare. Next to the 5-ft-diam opening for the tank was a 2-ft manhole opening. The tank dead load plus live load was 40,500 lb. With most of the slab consisting of openings, a 6-in.-wide beam on one side of the tank would have to support 20,000 lb for a clear span of 8 ft. In looking at alternative air stripper tank designs, a chemical engineer told me that we could use an air stripper tank with a side air-entry opening in lieu of a bottom air-entry opening. The tank would be 4 ft taller but only an 8-in.-diam opening would be required in the supporting slab. As a result, the structural slab width supporting the tank of the side next to the manhole opening was 3 ft, and a 12-in.-deep slab with reinforcing consisting of #7 at 6-in. spacing was used.

Wind loading versus seismic loading

Wind loading controlled over seismic loading as shown in Figure 3. In determining the center of gravity (CG) for the seismic loading, an engineer for an air stripper tank manufacturer was contacted for this dimension. In the absence of any value, the author suggested that one-half of the height of the tank be used for the tank's dead load and that one-third of the height of the packing within the tank be used for the tank's hydrostatic loading. The engineer stated that the assumption sounded reasonable and would suggest a different approach or value if his research into this matter indicated otherwise. The engineer never made such a response, but even if the CG of the tank's loads were at midheight, wind loading would still control over seismic loading as shown in Figure 3.

Foundation design

The anchor bolts in the peripheral base ring at the bottom of the tank were designed for a combined axial load plus bending moment. The moment of inertia of the bolt pattern was used in determining the maximum pullout force of the bolts. Overturning on the 9.33- by 17.33-ft foundation for the air stripper tank was checked assuming no restraint from the adjacent clearwell structure (Figure 2). Assuming base friction and no resisting soil pressure, the resultant fell within the middle one-third of the base with a maximum soil bearing pressure of 970 psf versus an allowable 3,000 psf. Then all of the horizontal load was assumed to be resisted by the soil pressure with no base friction. The resisting soil pressure increased by 30 psf to 150 psf which is far below the allowable passive pressure of 266 psf. The pressure is just above

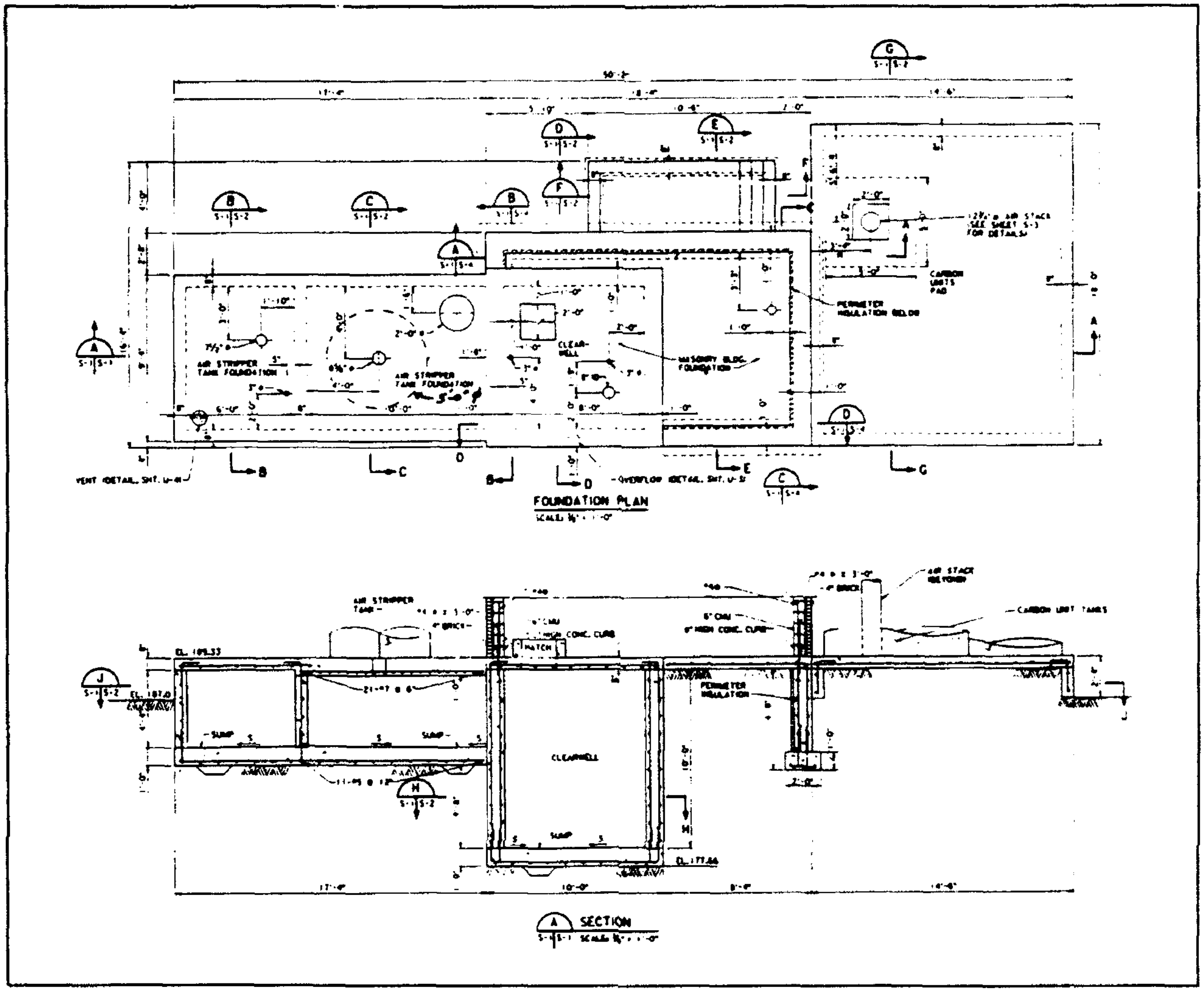


Figure 2. Plan and section of water treatment system

at-rest pressure which is desirable, since the movement associated with developing passive pressures is not tolerable. Joints are discussed in paragraph 5B.

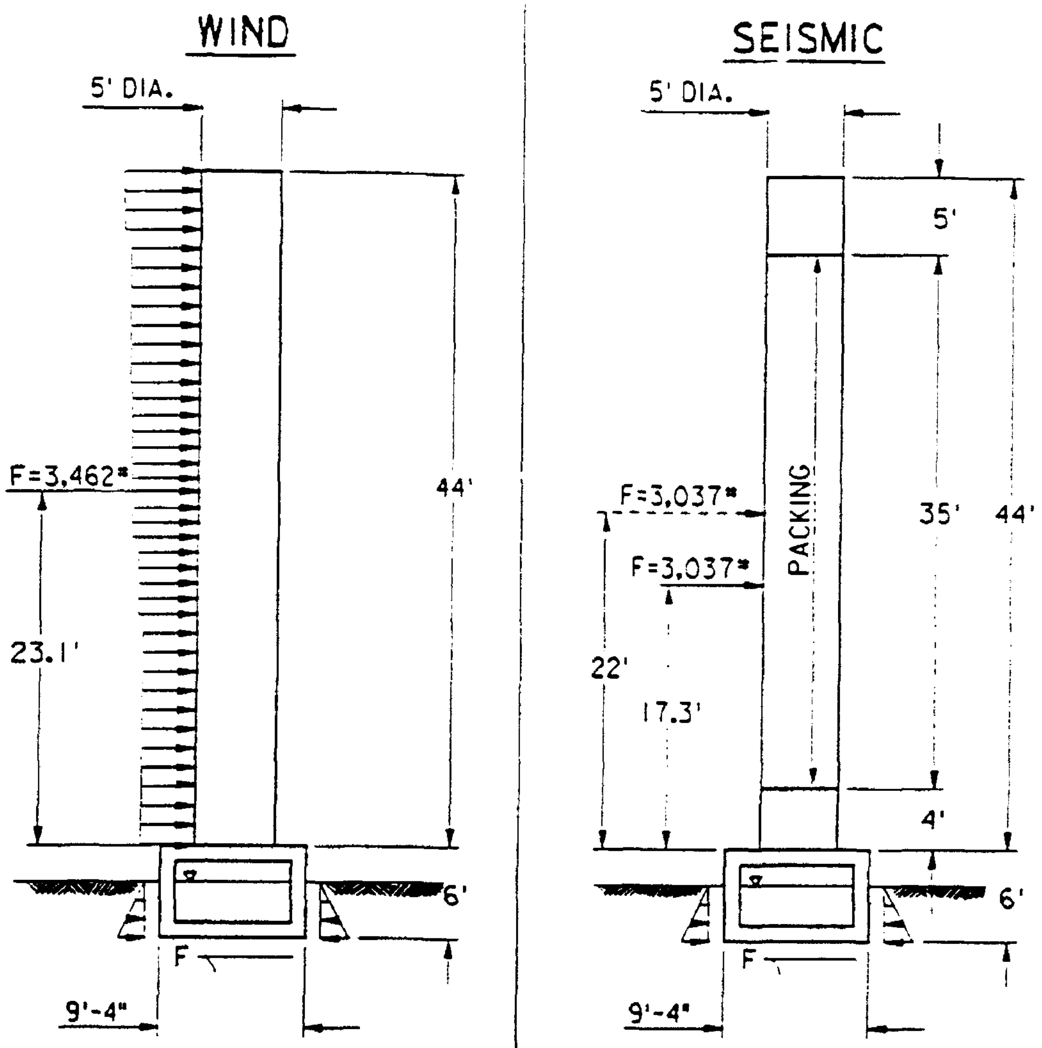
Design Considerations—Clearwell

Walls

The walls of the clearwell structure were designed as rectangular plates fixed on the sides and bottom and hinged on the top. Figures 13 and 14 from Moments and Reactions for Rectangular Plates, Bureau of Reclamation (Moody 1963) were used in the design of the walls. The inside of the structure was assumed to be dry, and an external load consisting of at-rest submerged soil pressure was used.

Joints

Watertight joints were used in the clearwell and air stripper tank foundation walls. These joints are roughened with reinforcing steel continuous through the joints with a thin flat metal waterstop as shown in Figure 4. This type of joint has performed satisfactorily on past Kansas City District dam and local protection-type projects.



$F = .00256\ k_z\ (IV)^2\ G_z\ C_f\ A_f$ ("Equation 1")
$F = \Sigma\ .00256\ K_z\ (IV)^2\ G_z\ C_f\ A_f$
F = 3,462#
C.G. of Load = 23.1' above tank base
Moment at Tank Base = 79,970 #ft.

$F = ZI\ 2/3\ C_p\ W_p$ ("Equation 2")
TM 5-809-10, Figure 10-3 (USACE, 1982)
F = 3,037#
$$C.G. = \frac{10{,}500 \times 22 + 30{,}000 \times 15.67}{10{,}500 + 30{,}000} = 17.3'$$
Moment at Tank Base = 52,540 #ft.
If C.G. at Tank Center = 22'
Moment at Tank Base = 66,810 #ft.

Figure 3. Air stripper tank, wind versus seismic loading

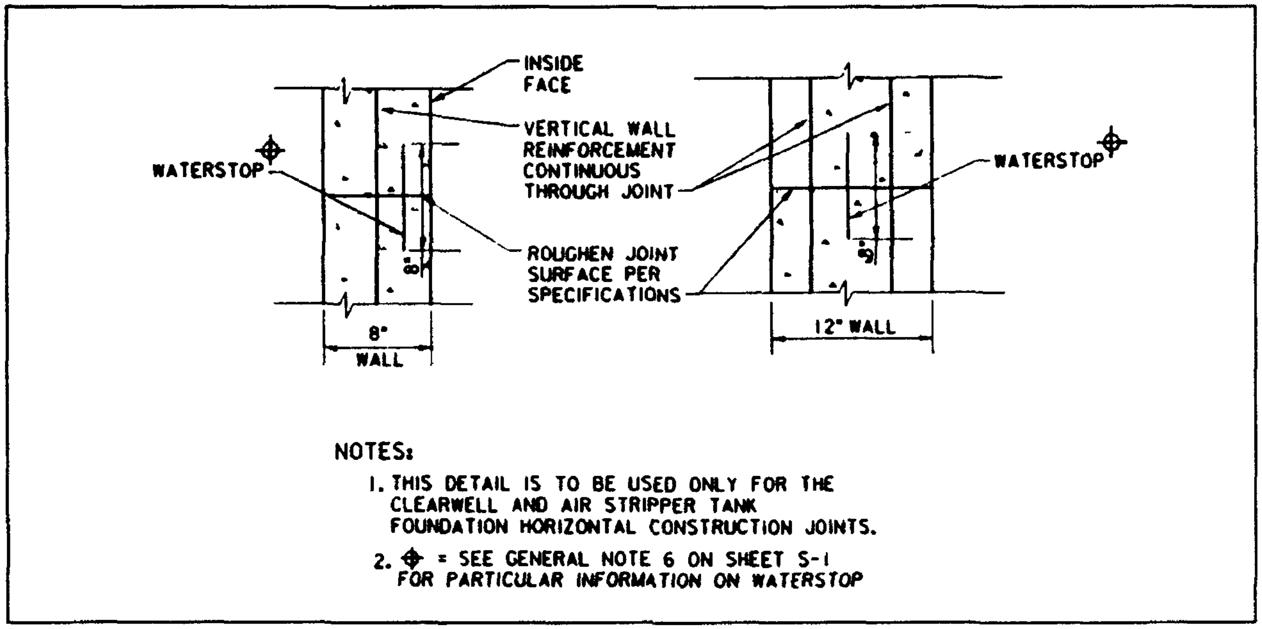


Figure 4. Horizontal construction joint detail

Design Considerations—Masonry Building

General

The masonry building for this project is 12 ft wide by 18 ft long by 8 ft high. The building completely encloses the clearwell structure (Figure 2) and will be kept at a temperature just above freezing. The building was originally preengineered metal, but the client opted for a masonry building to match the existing Well No. 7 building (Figure 5).

Foundation

Part of the masonry building is supported on the clearwell walls and the other part on continuous footings (Figure 2). Normally, 2-in. rigid perimeter insulation is placed on the inside of all the foundation walls down to the footing, but in this case, since some of the clearwell walls serve as foundation walls and could be filled with water, perimeter insulation was placed as shown.

Walls

The walls will consist of an exterior brick wythe and an interior CMU wythe (Figure 5).

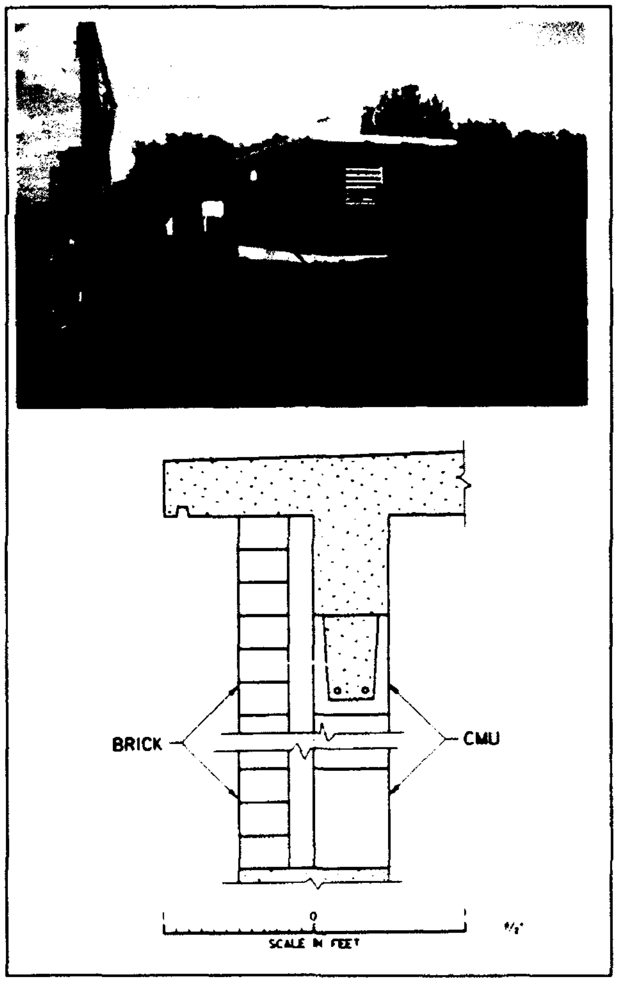


Figure 5. Masonry buildings (existing and section through new)

The walls were designed as anchored veneer, as per paragraph 8-19 of TM 5-809-10 (Headquarters, Departments of the Army, Navy, and Air Force, 1982). The brick wythe is nonload bearing with shear and vertical loads being carried by the CMU wythe. The cavity will be filled with insulation.

Roof

A reinforced concrete roof was designed to bear only on the CMU wythe. A watertight compressible joint was placed on top of the brick wythe to keep moisture out of the cavity yet not allow the concrete roof to bear on the brick wythe.

Design Considerations—Air Stack

Pipe diameter

The air stack is 40 ft tall (Figure 7). Due to site location, guy wires could not be used and it was undesirable to provide intermediate bracing to the adjacent carbon unit tanks. The air stack was designed as a cantilevered structure. The mechanical engineer wanted a pipe diameter ranging from 8 to 10 in. In considering the slenderness ratio of the pipe, a 12-in. diameter extra strong pipe was required. The mechanical engineer said that the increased pipe diameter would effect the rate of exhaust, but that this was tolerable.

Foundation

The base of the air stack is welded to an embedded plate (Figure 6). In determining the weld size to use to connect the airstack to the base plate, *Design of Welded Structures* (Blodgett 1966) was used. From Table 4 of that publication, the peripheral weld around the pipe was treated as a line force to transmit the bending load acting on the airstack. The section modulus for the weld treated as a line was given in Table 5 of the referenced publication. It was determined that a 1/4-in. fillet weld could carry the load. The anchor bolts in the base plate were designed for a combined axial load plus bending moment. Wind loading controlled over seismic loading. The foundation was designed to resist the wind-load overturning moments and sliding forces using the same procedure as for the air stripper tank outlined in paragraph 4.C. Again the resultant fell within the middle one-third of the base, and the resisting soil forces were just slightly bigger than at rest pressures.

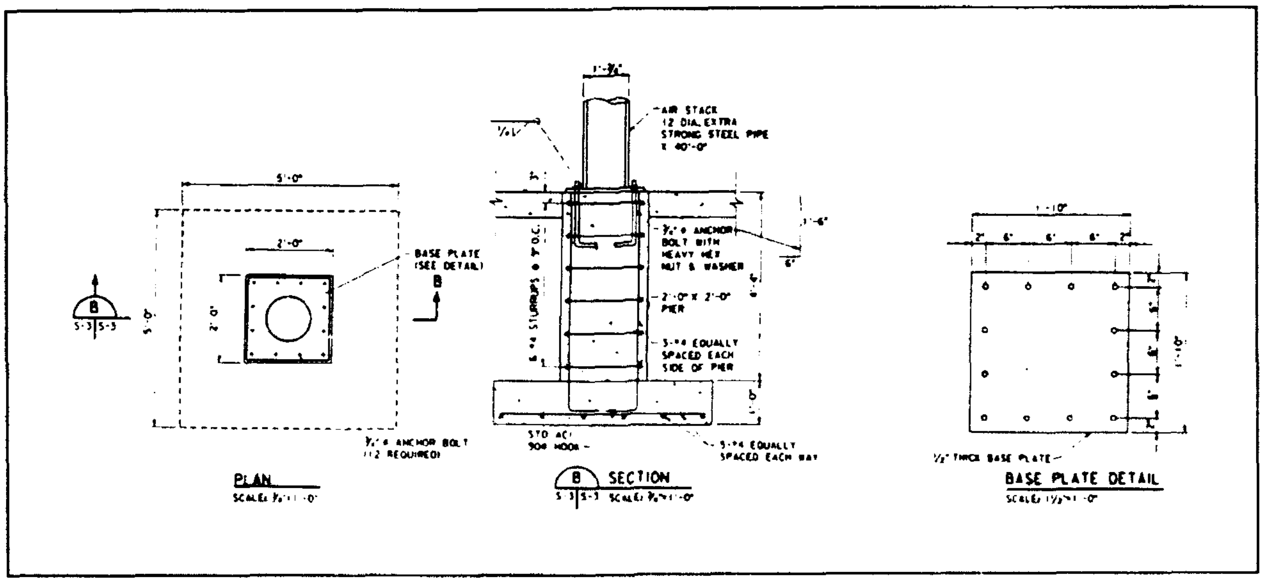


Figure 6. Air stack foundation details

Conclusions

As in the design of any project, the design of the water treatment system for Caldwell Trucking Well No. 7 required that different elements within this Superfund project be able to support the applied vertical and horizontal loads. Current military and civil works guidance were adapted for use as the design criteria for this superfund project. Past civil works and military project design experience and practices were utilized in the design considerations for this project. The input of the structural engineer at an early stage of the design process is essential to ensure total design quality. Structural considerations did alter the type of air stripper tank used, as well as changing the geometry of the air stack. The superfund program is a large developing program, and I believe that as structural engineers we can meet the design challenges that this program offers and produce designs that incorporate total design quality by carefully considering all of the structural aspects, by remembering and applying our past experience from civil works and military programs, and by making our presence known on the design team.

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Ben Sawyer Bridge Restoration

by
Mark H. Nelson, PE[1]

Shortly before midnight on 21 September 1989, category four Hurricane Hugo made landfall near the city of Charleston on the South Carolina Coast. One of the most visible and critical casualties of the hurricane was the 240-ft-long, 500-ton steel and concrete swing span of the Ben Sawyer Bridge. The force of winds from the hurricane caused the swing span to separate from its pivot support and begin sliding into the Atlantic Intracoastal Waterway (AIWW). Fortunately, the saddle portion of the pivot mechanism embedded itself in the pier. Instead, the 500-ton span was left with one end mired in the waterway and the other end high in the air while resting precariously on the outside edge of its hollow support pier. To complicate matters, the obvious inability of the bridge to carry traffic severed the only means of overland access to both the Isle of Palms and Sullivan's Island.

On 23 September, the US Army Engineer District, Charleston, mobilized damage assessment teams; one of which reported on the damage to the Ben Sawyer Bridge. The main truss members did not appear to have suffered any permanent damage. Some below-deck, cross-bracing angles had been bent or severed when they were struck by the pivot pier or mechanical equipment. Overall, the swing span was not deformed and appeared that it could be successfully reused. The highway department asked for technical assistance after inspecting the bridge. Concurrently, the Charleston District set out to evaluate other alternatives. A solution was needed that would quickly restore traffic flow to the islands and allow concerned residents to return to their devastated homes. On 24 September, the most promising alternative of building a combination floating ribbon bridge and marsh road was studied and discarded. The resources required to exercise such an alternative were judged to exceed the effort required to set the swingspan back in place.

On 25 September, the Charleston District developed a plan to restore the Ben Sawyer Bridge to a fixed position. As-built drawings were not immediately available, so truss member sizes and overall swing span dimensions and weight were estimated based upon visual observations. The estimated weight was within 10 percent of the actual design weight. Floating plant capacity available in the Charleston vicinity was considered, and it was decided that the necessary equipment was available to lift and place the swing span. Lifting strategies and connection points were evaluated to ensure that the swing span could survive the operation. Later that day, the state coordinated with the Federal Emergency Management Agency (FEMA) to task the Corps with restoring the Ben Sawyer Bridge to a temporary fixed position. FEMA did so the evening of 25 September. The District immediately prepared a request for proposal (RFP).

The RFP required the contractor to provide calculations and sketches for all proposed actions to be reviewed on site by district structural engineers. The RFP also required the contractor to maintain the existing structural condition of the swing span and to repair any damage caused by his operation. The district scheduled construction inspectors and structural engineers to provide 24-hr-a-day support if needed for the operation. The engineers were briefed on critical considerations that required particular inspection. A combined design/construction inspection effort was developed that allowed a flexible and immediate response during the emergency restoration of the swing span.

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[1] *Structural Engineer, US Army Engineer District, Charleston; Charleston, SC.*

At 1000 on 26 September, the Corps held a prebid conference with interested contractors at the bridge. RFP packages were issued to each contractor, a review of the contract was conducted, and contractors' questions were answered. The contractors were then instructed to submit their proposals no later than 1600 that day. To assist the proposers, district representatives made themselves completely available to answer questions and support the proposers in any way possible.

The restoration method proposed by the contractor involved the use of a large floating crane to tilt the swing span back to a horizontal position. Two floating barges with steel frame support structures would be used to float the span by rising with the incoming tide. The span would then be moved into place and set back on the support shoes by flooding the barges. This procedure did not require the span to be lifted free from overhead but actually supported reswing span from underneath. This provided a more stable operation with less potential for damage or accidents.

The Coast Guard initially opposed temporarily fixing a span over the AIWW but eventually agreed. On 26 September, two proposals were opened at approximately 1830. A rapid but thorough evaluation of both proposals yielded the conclusion that both technical approaches were not only acceptable but very similar. A contract award was made to the low bidder, Hardaway Company, for $248,400. A notice to proceed was prepared and issued by 2030 on 26 September.

By 27 September, the Contractor was fabricating custom framework for the project. On 29 September, the contractor was on site and continuing preparation. Although weather delays and equipment problems plagued the contractor, the bridge was placed back into position by 0300 on 6 October. The bridge was then jointly certified for safety by the Charleston District and the South Carolina Highway Department. On 6 October, traffic was crossing the bridge by 1445.

Throughout the entire operation, the district worked closely with the highway department, the Coast Guard, and municipal officials to coordinate the myraid of details necessary to preserve waterway safety, support the contractor, and provide private citizens access to their island residences.

It is worthy to note that the Corps accomplished the Ben Sawyer Bridge Mission almost 7 weeks quicker and at less than one-half of the original proposal cost independently presented to the highway department by a private contractor. Finally, the mission was accomplished without a single injury of any type. Recognizing the magnitude of risk associated with a job like the restoration of the Ben Sawyer Bridge, we are proud of this accomplishment and everyone involved.

Ben Sawyer Bridge restored

Structures in the Blue River Channel Kansas City, Missouri

by
Morris E. Ganaden, PE[1]

Abstract

This paper provides a discussion on structural features of a channelization project located in the Kansas City, MO, area. The Blue River channelization project (approximately 12 miles in length) will control upstream flooding for large residential and commercial areas along the Blue River plain. The structures to be discussed are the following:

An A-E designed concrete grade control structure will be located at 58th Street, near the upstream end of the project, to dissipate the river's energy. The 118 ft wide by 169 ft long basin is designed to pass a 500-year flood of 35,000 cu ft/sec.

The Corps designed Paved Reach will consist of approximately 3,500 ft of Blue River Channel to be paved downstream of Independence Avenue (9th Street), in order to provide hydraulic efficiency through the Armco Steel Mill industrial area.

Numerous concrete structures are required in the Paved Reach in addition to the paved channel. Among these is a 6-ft deep by 15-ft wide low flow flume which will be constructed using sheet piles with concrete floor, or precast concrete U-sections as per the contractor's option. A concrete slope stability structure, supported on H-piles, will be constructed to support the steel mill railroad storage and switching yard, providing relief for the unstable right bank of the river. An L-shaped concrete slab bridge, supported on H-piles, will be constructed to transport truck and railcar traffic for the Armco Plant. A 1,612-ft long concrete flood wall will be constructed to keep 30-year flood flows within the channel banks. Several other concrete structures in the Paved Reach are the drainage extension for the existing 17-ft horseshoe conduit, discharge structures for the 43 existing pipes (2 to 48 in.) that extend into the paved channel, and concrete boxes that contain sluice gates for control of discharge through the floodwall.

Some of the factors making the Paved Reach a difficult project to construct are dewatering of the channel area and diversion of the river during the 3-year construction phase will be difficult. The natural slopes of the channel are highly unstable and will require "pinning" with hundreds of closely spaced H-piles. The construction area is very restricted because of the Armco Steel Mill operation on each channel bank and will require special construction techniques to ensure compliance with right-of-way agreements. Also, large deposits of steel mill slag are

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buried in the river banks, making pile driving difficult. Chemical pollutants (PCB's) located in sections of the river bottom are being removed prior to construction of the Paved Reach. Four bridges span the river in the Paved Reach causing difficulty in the paving and other construction operations.

Introduction

The 12-mile long Blue River that meanders through the east side of Kansas City, MO, has caused periodic flooding of the residential and industrial complexes since that area was first inhabited (see Figure 1). The project was authorized by Congress under the Flood Control Act of 1970. The city participates under a cost-sharing agreement whereby all rights of way and relocation costs are the sponsor's responsibility. The plan resulting from the study and model testing of the basin was to rechannel, providing slope protection and upgrading existing bridges and other structures as necessary to provide the required 30-year flood protection. Approximately 207 million dollars is required for nine major construction projects, varying from channel straightening to massive numbers of steel H-piles to stabilize the extremely unstable channel slopes. Rock protection for side slopes and piers for the 41 bridges located in the river's flow path is also a large part of the overall reshaping of the existing river channel.

Grade Control Structure

A six million dollar concrete grade control structure will be located at 58th Street, near the upstream end of the project, to dissipate the river's energy during massive flood discharges. The concrete grade control structure is currently being designed by an A-E firm and will be constructed in 1997. The 57 ft-high basin walls are founded on rock. The 118 ft wide by 169 ft long basin has eight 7.5 by 7.5 ft baffle blocks to dissipate the discharging river flows. The river drops 10 ft, and the structure is designed to pass a 500-year flood of 30,000 cu ft/sec. The normal river flows will pass through the 5 by 13 ft notch in the basin spillway. The side walls presented some difficulty in the design. These are located on sloping footings. It was determined that a differential settlement of 2-7/8 in. from one end of the 120-ft-long (consisting of four monoliths) footing to the other end would be experienced. At first the use of H-piles was considered, but the lower monolith was founded on 6-ft H-piles, which according to AASHTO are inadequate in length since 12 to 20 ft of embedment is required to maintain stability. It was determined that if three rows of piles are used, the stability is not a problem. As long as three reactions are provided that are nonconcurrent and nonparallel, the system is geometrically stable. Even though it was structurally adequate, economics prevailed, and it was determined that as long as the footing is continuous, providing monolith joints only in the wall, we could accept the 2-7/8 in. settlement over the 120 ft length of footing.

The Paved Reach

The "Paved Reach" is a 3,500-ft length of channel, near the downstream end of the project, that extends through an industrial complex area that due to the bridges and channel alignment will require paving. Numerous concrete structures will be constructed to ensure that the Armco Steel Mill industrial plant is not adversely affected by the channelization (see Figure 2). One of the main problems that makes the Paved Reach difficult is the many Armco Steel Mill buildings and railroad operations located on both banks for the entire reach. Although generally cooperative, mill officials were adamant about maintaining their operation capability during the construction of the Paved Reach. This involved serious negotiation for minimal temporary easement and right-of-way for constructing the pavement and other structures. Construction access was particularly difficult, considering that the mill operations were not to be disturbed while the construction phase was underway. The contractor would

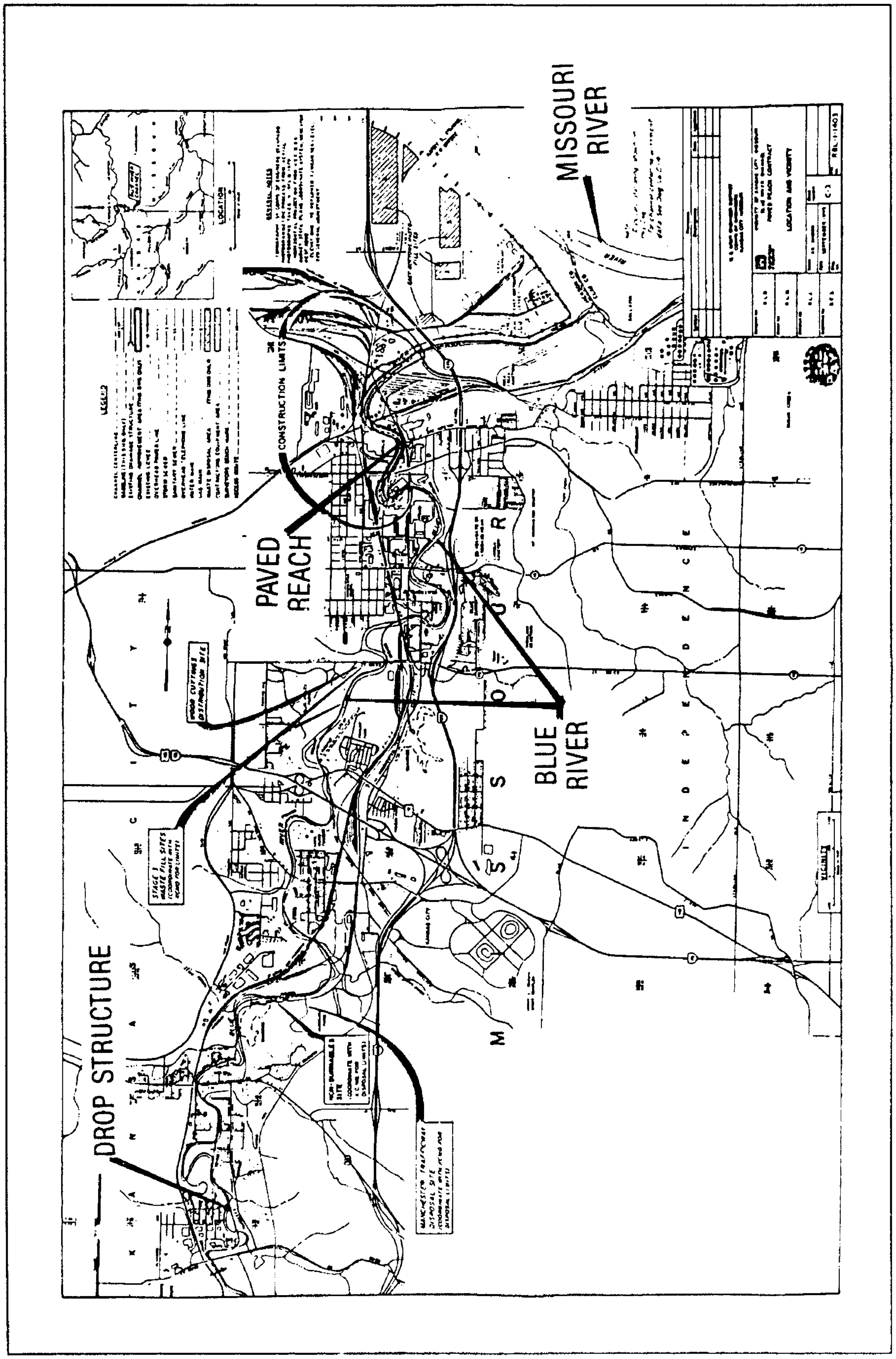

Figure 1. Blue River, periodic flooding area

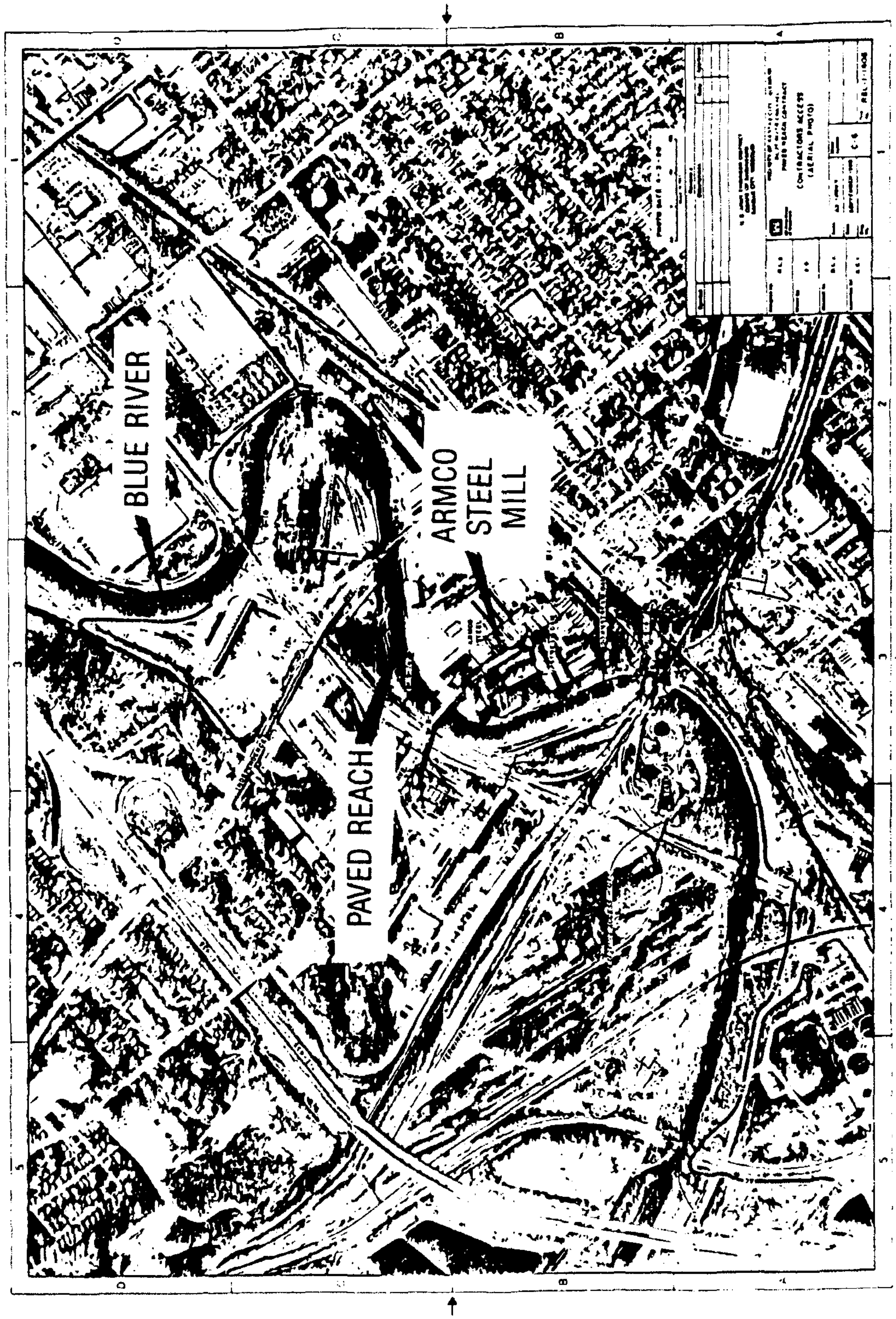

Figure 2. Paved Reach through industrial complex area

be required to notify the mill in advance of each operation so that the mill could work around the construction schedule. In some areas the limited construction easement and ROW would require that sheet piling be driven in order to not violate the mill's property limits. The coordination of the project was through the city of Kansas City. Armco Steel Mill and other private owners would be contacted by the city concerning the operation of the four bridges involved in the reach; two are owned and operated by the mill and the other two are owned by private railroad companies. Work to be done to the bridges to reflect the channel changes was to be conducted by the bridge owner and reimbursed by the Government.

Other problems encountered in the Paved Reach are the soil in the existing channel reach being saturated with steel-like slag left over from the steel processing operation and dumped for over 40 years by the mill, to protect and stabilize the banks. This slag varies in size from baseball to automobile-size chunks that are located up to 20 ft deep in the embankment. Also, the channel side slopes, as experienced in the upstream reaches, are highly unstable. Armco has had to deal with the serious slides of the embankment for many years. The Corps stabilized the banks by driving hundreds of H-piles for a pinning action. This was successfully used for other channelization contracts upstream from the Paved Reach contract. In addition, the Blue River had a serious deposit of PCB's located upstream of a dam constructed in the channel by Armco to obtain cooling water for their plant operations. The PCB's were removed by separate contract prior to the Paved Reach contract which was awarded in February 1991. Probably the most important factor to consider during construction is that severe flooding has been experienced in the reach almost on a yearly basis.

Some of the concrete structures that would be constructed in the reach are 3,500 ft of trapezoidal-shaped channel with a 15-ft wide by 5 ft-6-in. deep low flow flume constructed in the invert. A concrete gravity-type flood wall will be constructed for 1,612 ft along the left bank to provide the mill with 30-year flood protection. The wall will be constructed with two 48 in. drainage conduits and concrete drop structures with sluice gates to allow for drainage of the water from behind the wall. A buried concrete slab supported on H-piles will support the mill's railroad yard on the right bank. An L-shaped concrete support structure on the left bank will carry Armco Steel Mill truck and railcar traffic at an important access to the mill. Extension of an existing 17-ft horseshoe storm discharge conduit is to be constructed on the left bank that is skewed between two existing bridges. Other concrete structures are the headwall and extension structures for the approximately 43 existing drainage pipes emptying into the channel. Some interesting details regarding the individual structures of the Paved Reach are as follows:

Channel pavement

The 8- to 10-in.-thick pavement will follow Corps EM guidelines and will be jointless with the exception of construction joints and isolation joints located at structures, and will have No. 6 bars at 12 in. centers in each direction. According to the EM design criteria, pavement contraction and control joints cause problems in maintenance and the No. 6 bars will keep cracks to a minimum width. Prior approved construction joints provided in the pavement at the end of a construction period will be allowed. The slab is designed to support construction and maintenance equipment vehicles.

About 4,000 psi compression strength concrete will be used for the pavement. It is expected that large pavement-placing machinery similar to highway equipment will be used for expedience and economy. The basin slab subfoundation is drained using 6-in. PVC subdrains equipped with C.I. flap gates and a 12-in. collector pipe. The flap gates ensure that silt does not accumulate beneath the slab causing blockage of the subfoundation drainage system, which could result in uplift forces and buckling of the pavement slab. Built into the one on three side slopes of the pavement are stairs at locations where monitoring of the existing discharge pipes will be conducted.

According to PCA criteria, if the pavement foundation subgrade coefficient of friction is acceptable, adequate reinforcement is used, and construction joints are provided, then large areas of slab can be poured without a joint. The drainage system beneath the pavement has cleanouts located at 200-ft intervals so that future maintenance will assure that the drains are not blocked. The PVC drain pipe will be used with ductile iron fittings.

Low flow structures

In order to provide maximum economy and flexibility of the contract, two alternate low flow schemes were provided in the contract plans and specifications for the contractor's bid. The sheet pile scheme provides a 15 ft wide by 5.5 ft deep flume. The invert is constructed using a 1 ft-6-in.-thick cast-in-place concrete slab that acts as a strut between the sheet piles. The concrete slab is doweled to the sheet piles and provides lateral support, reducing the moments and deflections in the sheet piles to within tolerable limits. A special pump-operated PVC pipe drainage system is located in the subfoundation beneath the slab in order to reduce uplift pressure during construction of the slab. The sheet pile scheme will be constructed in a sequence that will provide diversion of the river to either side of a line of sheet piles. The first line of piles will be driven beneath the river flows. A sheet pile test section 184 ft in length was completed in an earlier contract that was let in order to remove the PCB's from the river bottom. No problems were encountered while driving the test section sheet piles. An interesting development involving the sheet pile scheme was encountered after the contract was advertised. As for the contract specification, only hot rolled type sheet piles were to be used. Sheet piles using light weight cold formed steel sections were considered inadequate. The manufacturers of cold formed sheet piles hit the ceiling. During earlier discussion we informed the cold formed people that the main reason for rejection was that the joint connecting the sheet pile sections was inferior and would become "unzipped" while driving. After several telephone discussions, assuring us that their joint was structurally sound, and had been proven by actual testing, and after a letter from a congressman in the manufacturers' home state, we decided to amend the contract to allow cold formed steel sheet piles. Other properties such as strength and corrosion resistance were as good or better than the hot rolled members. Drain holes with cast iron flap gates are to be located at 5 ft centers in the sheet piles. The flap gates are necessary to resist the infiltration of silt laden waters into the channel rock fill. The intake and outlet ends of the low flow channel will have a 125-ft sheet pile transition into the channel section. The low flow alternative is the use of approximately 700 precast concrete U-sections approximately 5 ft in length with a 15 ft wide by 1 ft thick invert and 4.67 ft high by 1 ft thick walls. The 5 ft sections weigh 22,800 lb and will require a 100-ton crane to position the sections. These will be set in a 1 ft bed of crushed rock. The primary concern in the U-flume alternative is that these sections will require placing in the dry, and may require constructing dams and dewatering and watering up the area almost daily to provide a dry construction environment while maintaining river flows The Blue River flood record is very sporadic and the possibility of wipe out during the summer floods is a hazard. The precast alternative will require straight and curved sections to follow the variable alignment of the channel. A number of precast companies in the area expressed interest in furnishing the large number of these units (700). The weight of the precast units must be maintained in order to prevent uplift. The contractor's precast manufacturer indicated that the precast U-sections would be constructed by first pouring the 16 ft by 5 ft by 1 ft slab, then after two days, curing, using high-early strength additives, the side walls would be cast. He "figured" on casting approximately seven units per day. An interesting item of note in the competition between the low flow sheet pile scheme and the precast section scheme is that in the actual bidding of the Paved Reach contract, where both schemes were offered for bid, the U-flume scheme was the only scheme bid on; i.e., all

seven contractors chose to bid on the precast U-flume scheme rather than the sheet pile scheme. This was a surprise in the Kansas City District since most felt that the sheet pile scheme would be the most economical because of complicated dewatering and possible flooding of the channel. A reason given for selecting the precast U-section scheme is the possible difficulty in driving the sheet piles. Although a test section was driven, the slag-laden area still presented some unknowns in the use of sheet piles, since the piles are "stopped," even if a buried log is encountered. The possibility of other debris in the channel bed and the difficulty in using pile-driving barges "scared-off" the contractors from bidding on the sheet pile scheme (see Figure 3).

Slope stability structure

The slope stability structure that was originally planned was to support a railroad switchyard approximately 65 ft in width and 1,230 ft in length. Several schemes were considered, from a buried cast-in-place slab, timber structure with steel beams, and a buried precast prestressed box section system, all supported on concrete caps with steel H-piles. The precast, prestressed box system was finally selected, and the completed scheme developed for the plans and specifications. According to Armco Steel Mill negotiations, the contractor would be limited to 6 months for the construction of the stability structure. The difficulty with the precast system was the unit weight and that access to the structure would be difficult, considering the mill's stringent ROW limits. It was finally decided that the precast stability structure would have to be partially constructed by transporting the units using a 60-ton crane on top of the previously positioned precast units as they are set on top of the poured in place concrete pile caps. The design of the precast box units would be accomplished by the contractor. Controlling design information for the precast, prestressed box units shown on the contract plans and specs was received from the Santa Fe Railroad Company that had built similar bridge structures using precast box units. The railyard support structure was designed for Cooper's railroad loading and for construction loads using a 60-ton crane with loaded precast units. Later, approximately 3 weeks before the Paved Reach contract was to be advertised, Armco Steel Mill officials indicated that they would relinquish the need for the large railroad yard support structure for a much smaller structure to support two railroad tracks for a shorter distance. The last minute design resulted in a 14 ft wide by 300 ft long, cast-in-place concrete slab support on H-pile caps at 14 ft centers. This resulted in a much simpler structure. ROW was no longer a problem since the structure was smaller, and equipment for the cast-in-place concrete operation would have easy access to the structure. The reason for the mill's change in plan was primarily a change in mill operations that would not require the need for the large railroad switchyard storage area. They also were compensated by the city for future maintenance that would be required for the original stability structure and other cost to the city that would amount to approximately 1 million dollars in savings for the mill (see Figure 4).

Support structure

Another bridge-type structure is the concrete railroad switch and truck support structure located at the corner of the ball mill building on the left bank of the channel. This location is of primary concern to the mill in maintaining operation for this building. The bridge is a slab design, 1 ft thick, supported on H-piles, providing a continuous four span and two span L-shaped structure around the corner of the building. Trash fenders fabricated from steel wide flange beams are provided to keep trash from building up beneath the structure. Adequate expansion joints are located in the slab to allow the structure to move with temperature change. The original design concept at this location was a sheet pile, earthfill type structure, but the possibility of slag in the embankment and the difficulty in driving sheet piles caused this scheme to be abandoned. H20 S16 truck loading and Cooper's railroad loading were used for design of the support structure. The contractor is required to construct the support structure in a 6-month time restriction

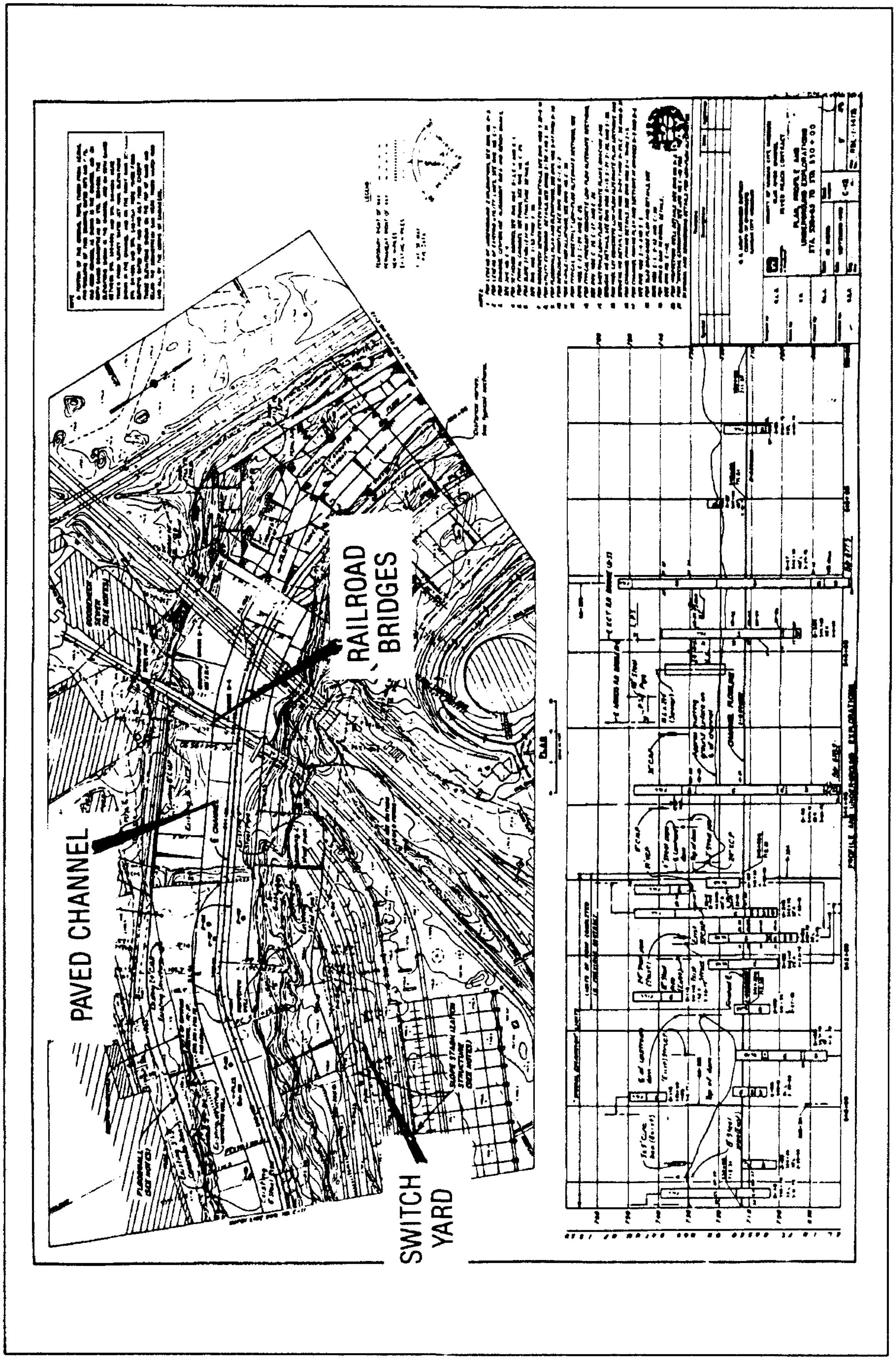

Figure 3. Low flow structures

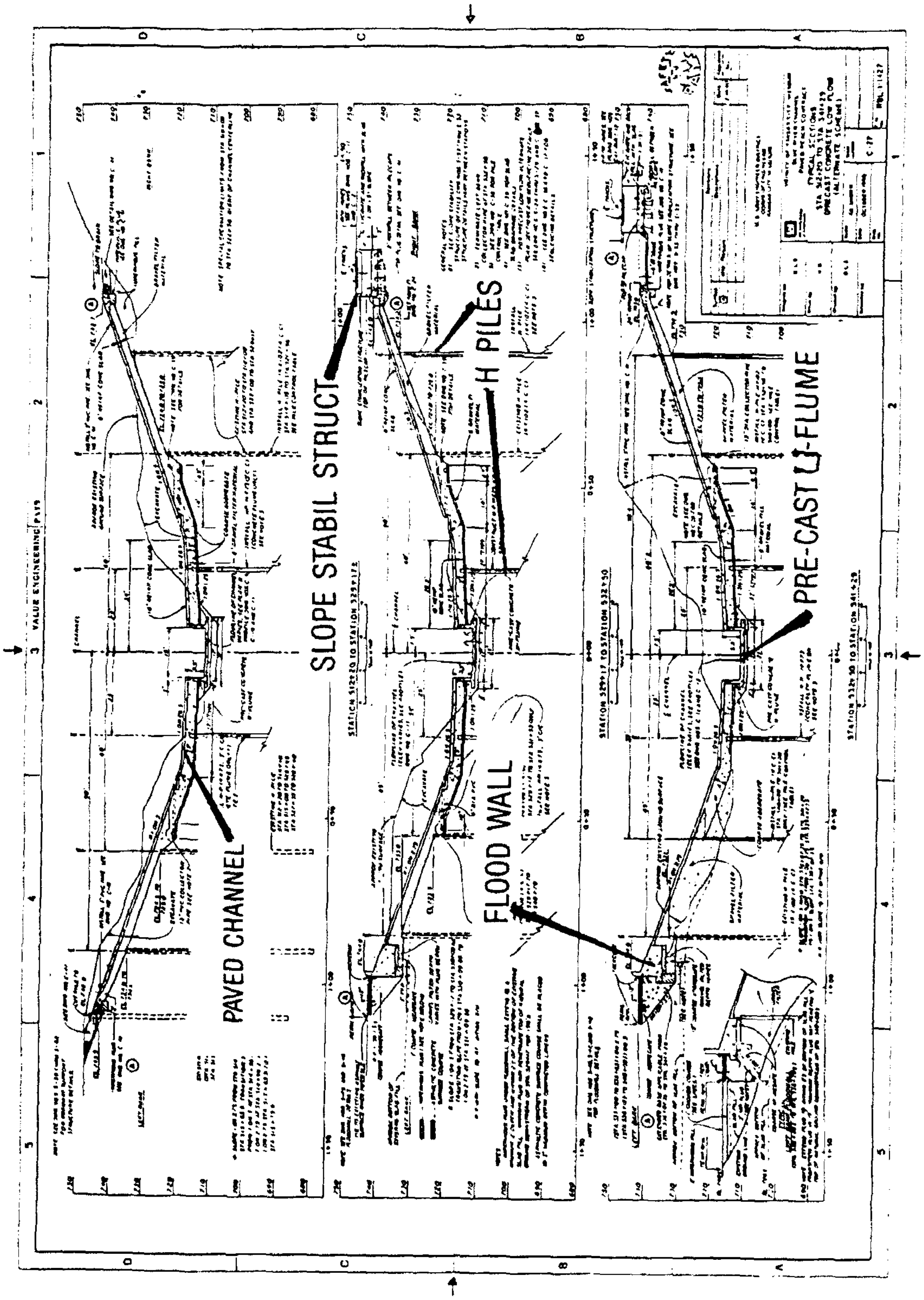

Figure 4. In-place stability structures

Flood wall

The 1,612-ft long flood wall constructed on the left channel bank will be capable of withstanding a 30-year flood. The concrete wall will be a gravity type and will have an 8- to 10-ft wall height. An impervious layer beneath the wall will prevent river waters from migrating to the land side of the wall. The wall will have 30 ft monoliths and will be designed for a maximum 6-ft differential head from the river side and a maximum 4-ft differential head from the land side, considering rapid drawdown of the river after the wall has been overtopped. The primary problem in the design was sliding toward the river as the result of the landside differential force. This resulted in a larger wall section to provide the necessary weight and base friction force. An asphalt road covering on the landside of the wall helps to drain the landside of the wall. Two large 24 in. drainage pipes project through the wall and are equipped with flap gates and sluice gates for emergency backup. The wheel-operated sluice gates are located in concrete drop structures. The flood walls will be reinforced for temperature and shrinkage and 4,000 psi concrete will be used. Ladders will be located over the wall at stair access locations on the Paved Reach. The top of the wall will be tapered in order to prevent people from walking on top of the wall. The river side of the flood wall will be sloped to match the paved channel surfaces.

Goose Neck Creek drainage structure

The existing Goose Neck Creek drainage conduit that discharges into the channel in the Paved Reach section will be extended with a 16- by 21- by 70-ft concrete box and U-wall section that will be supported on H-piles. The existing conduit was built in 1929 and consists of a 17-ft-high horseshoe conduit with concrete headwall. An expansion joint will be located between the existing and the new structure. The existing head wall will be left in place and built up where spalling of concrete has occurred. The outlet structure will transition into the paved channel section, discharging into the low flow flume. A flattened, sloped section will be constructed across the outlet invert, in order to allow maintenance equipment access at this location. The conduit extension and outlet was designed with a full head of water on the walls and foundation, considering maximum flow through the conduit, while the river is at its normal flow line.

Utility extensions

Approximately 58 existing pipe conduits of various sizes (2 to 48 in. in diameter) and type, PVC, steel, CI, VCP, and concrete extend into the channel. The pipes that are active, discharge primarily water from the Armco Steel Mill operation; however, other unknown liquids have been observed to flow from some of the pipes. The pipes will receive various treatments, from pipe extensions to cutting and plugging if the pipes are no longer used. Approximately 43 of these will require extension and concrete head walls to provide continued services. The existing metal pipes will be required to be removed back to the ROW line or the limits of the concrete pavement in order to ensure access if the pipes corrode and require replacement. The metal pipe will be replaced with concrete or PVC pipe, depending on the size. The nonmetal pipes will be replaced with like kinds of pipe and construction. Access stairs will be formed into the pavement surface to approximately 26 of these pipes so that monitoring technicians with equipment will have access to the pipe discharges. The pipes will be fitted with a concrete collar at the intersections of the existing pipes. The specifications require that the contractor keep all the pipes operating during the construction of the reach. Also, it is expected that other unknown pipes will be uncovered during the construction operation and will require plugging. The latter was discussed in the Specifications. Each of the new pipe extensions will be fitted with a cast iron flap gate at the discharge end to ensure that flood waters do not back up through the intake end of the pipes. The intake location is unknown for most of these pipes.

A concrete U-shaped pipe conduit 6 ft by 4 ft by 25 ft will be constructed in the pavement

beneath the Independence Street Bridge. This box structure was requested by the mill in order to route future conduits beneath the bridge and to protect them from flood discharges. The conduit required sizing to prevent uplift of the box-shaped conduit.

Contract award

The Paved Reach contract was awarded in February 1991 and will take approximately 3 years to construct. Seven contractors bid on the contract and the bids varied from a high of $37,000,000 to a low of $21,000.000. The Government estimate was $31,000,000. The low bidder, who was considerably lower than the others, was awarded the contract. His low bid was apparently reflected in reduced cost to construct the pavement and the U-flume concrete installation cost. At present the contract is on schedule and the river has behaved admirably for the contractor's benefit. No unforeseen problems such as side slope slides, overabundant slag deposits or unworkable ROW access have been encountered to this date.

Sheet-Pile and Precast Concrete U-Flume Low-Flow Channels for the Blue River Paved Reach Project

by
Kurt A. Mitscher, PE[1]

Abstract

The Blue River Channel modification project is designed to provide flood protection to the Blue River Basin in the vicinity of Kansas City, MO. In an industrial reach of the river, the project consists of extensive modification of the channel cross section and construction of a floodwall, paving of channel-side slopes, construction of a 15-ft-wide by 5.5-ft deep low-flow channel for approximately 3,700 linear feet of the river. Weak clay strata and the existence of steel mill slag and other wastes soil have complicated project design. Due to concerns about existing conditions and the feasibility of pile driving through the existing slag, two complete alternate designs were prepared and incorporated into the project plans and specifications. The first design consists of steel sheet-pile walls with a cast-in-place concrete invert. The invert was designed to act as a strut to the sheet-pile walls. The second alternate consists of precast concrete U-shaped units. This study presents the details of both analyses, a discussion of which alternate the contractor selected, and any problems encountered during construction.

Project History and Purpose

The Blue River Channel modification project is designed to provide flood protection to the Blue River vicinity of Kansas City, MO. The Blue River passes through the length of the city flowing from south to north and discharging into the Missouri River. From approximately 13 miles upstream of the Missouri River to the mouth of the river, the Blue River flows through a heavily industrialized area. Flood control planned for this reach consists of extensive channel cross-section modifications, installation of piles to increase slope stability, construction of a 1,700-ft-long gravity floodwall, construction of a railroad bridge for slope stabilization, and paving of the channel with both rock fill and concrete. In the most constricted portion of the river, channel treatment consists of concrete paving and a 15-ft-wide by 5.5-ft-deep low-flow channel. Design and construction of the low-flow channel were complicated by the presence in the riverbanks of weak clay layers (with phi values as low as 11 deg) and fill placed by operations of a steel mill and other industries. The fill contained waste steel smelting slag and other debris. The existing soil conditions led to unstable bank slopes which have been stabilized by the installation of H-piles at 4-ft centers along the river. This work was performed under a previous contract in order to allow the river water level to be lowered during removal of sediments contaminated with polychlorinated biphenyls (PCB's). Additional H-piles are to be installed in areas which were found

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to be unstable after the completion of the H-pile installation contract. The design of the low-flow channel was also complicated by the existing weak soils and the slag in the fill. Due to the soil conditions and to concerns about constructibility of the low-flow channel, complete designs for two alternatives were prepared and included in the plans and specifications. The first alternative, which was included in the GDM, was a steel sheet-pile channel with a cast-in-place concrete invert, and the second alternative consists of a precast concrete U-flume. As part of a previous construction contract to remove contaminated sediments, a 180-ft-long section of the low-flow channel was constructed using the sheet-pile design.

Sheet-Pile Low-Flow Channel Design

The original design for the low-flow channel consisted of a pair of sheet-pile walls with a cast-in-place concrete strut between the walls. Tops of sheet pile were left approximately 2-1/2 ft above their final cutoff elevation to allow diversion of water around the low-flow channel. This diversion allows for construction of the concrete strut in the dry. The concrete strut is connected to the sheet pile with welded headed studs to account for transfer of uplift loads on the bottom of the concrete strut to the steel sheet piles. Details of the steel sheet-pile alternative are shown in Figure 1.

The sheet pile was analyzed using the CASE programs X0031 (CSHTWAL) and X0061 (CSHTSSI) and a SMART spreadsheet. CSHTWAL was used to determine the active and passive soil pressures. However, because the elevation of the strut was so far below the top of the sheet pile and the weakest layer of soil is above the strut, the top of the sheet piles tended to rotate into the bank, and the program would not continue to a solution. Instead, the soil pressures output from CSHTWAL were input into a SMART spreadsheet that was specially set up for this problem and analyzed using the free earth method. These results were then verified using the program CSHTSSI.

The CASE program CSHTWAL provides for analysis and design of sheet-pile walls using classical methods. The program assumes that full active and full passive pressures are developed at all points along the length of the piles. It also assumes that the top of the pile tends to move away from the soil behind it. Since the completion of the project design, this program has been modified to allow different factors of safety to be applied to active and passive pressures and to allow different factors of safety to be applied to different soil layers. The program has also been renamed CWALSHT. Unfortunately, all of the bugs have not yet been worked out, and the program is not yet usable.

The CASE program CSHTSSI provides for analysis of sheet-pile walls by modeling the soil-structure interaction of the system. It models the soil using nonlinear pressure-displacement curves which allow for pressures that are less than full active or full passive pressures. The nonlinear curves are generated by subroutines contained in CSHTSSI.

Analysis of the sheet-pile walls was performed for four design cases: Case I, Normal Operation with Completed Construction; Case II, Surcharge Loading with Completed Construction; Case III, Construction with Water Diversion; and Case IV, Construction with Crane Surcharge. The two construction cases assumed the top of sheet pile at elevation 721, and the two completed construction cases assumed top of sheetpiling at elevation 718.5. The required factor of safety for the two completed construction cases was 1.25, and the required factor of safety for the two construction cases was 1.00. Surcharge loading consisted of a 300-lb/sq ft strip load starting 8 ft behind the wall and extending an additional 15 ft. The strip load was intended to represent a maintenance vehicle traveling along the channel on the berm. Design soil profiles and assumptions for each case are shown in Figure 2.

For the two construction cases, the sheet pile was first evaluated as a cantilevered wall. The results showed that moments in the piles would exceed the moment capacity of all standard PZ

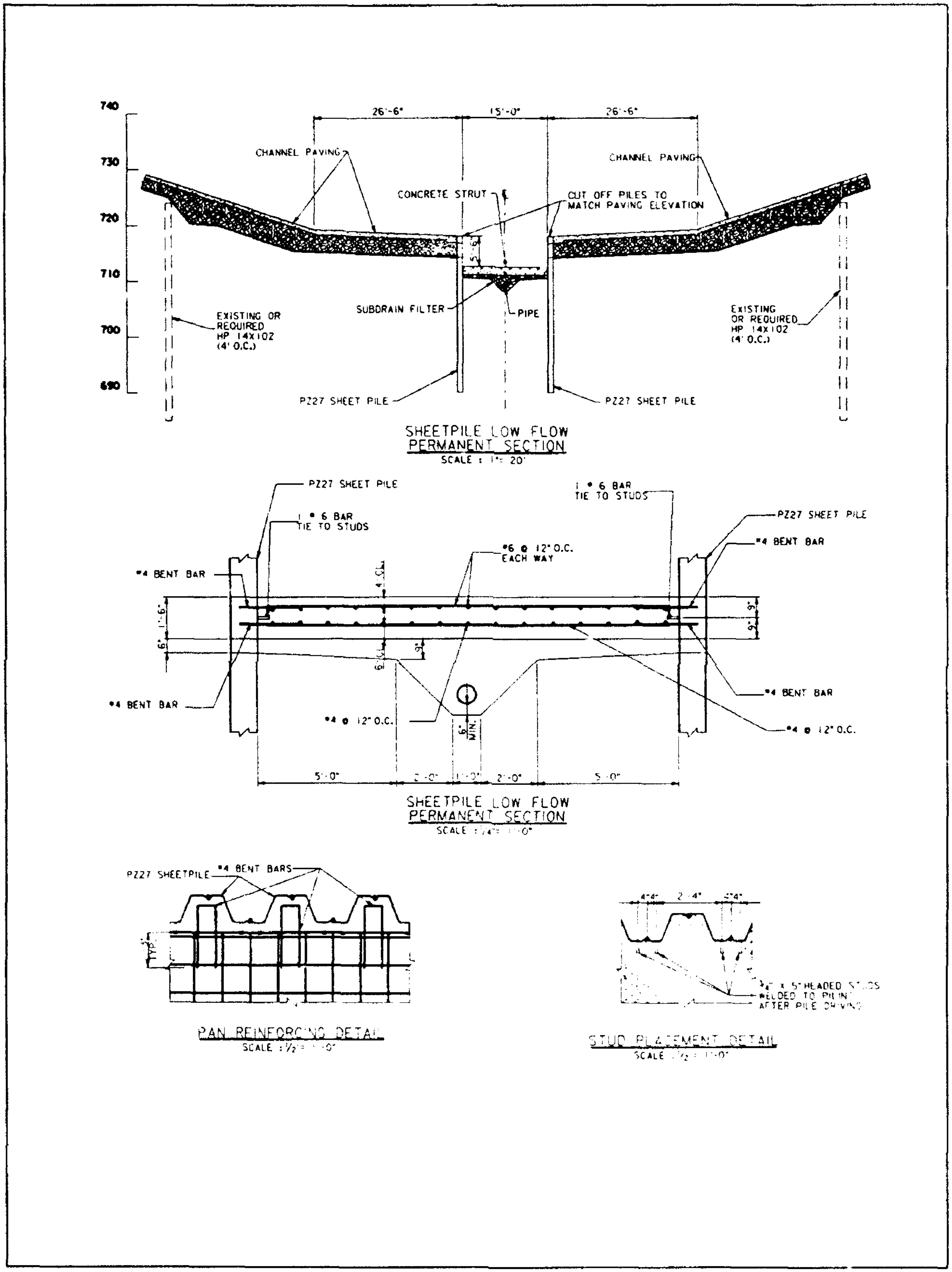

Figure 1. Sheet-pile low-flow typical details

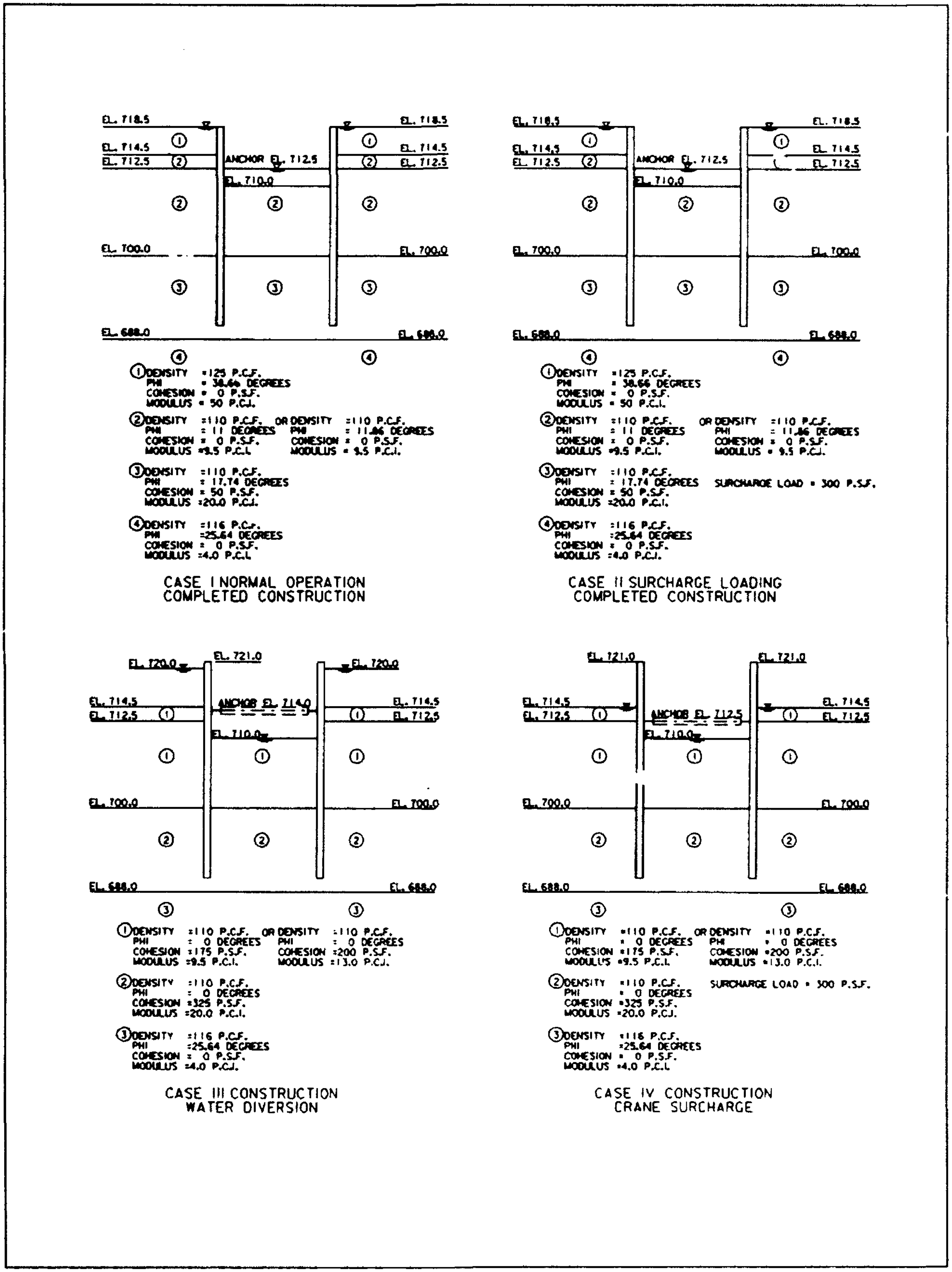

Figure 2. Sheet-pile low-flow design assumptions

sheet-pile sections. Sheet pile was then designed as an anchored wall for all cases, and temporary bracing was required until the concrete strut was completed. As mentioned above, the program CSHTWAL would not run because the strut elevation was so far below the top of the sheetpiling. This caused the top of the wall to rotate into the ground behind it rather than away from the ground as would normally be expected in a classical analysis. Since the top of the wall rotated into the ground, the sheet pile was evaluated twice, assuming first that active earth pressures acted on the piling above the strut as in a classical sheet-pile analysis, and then assuming that passive earth pressures acted on the piling above the strut. Both assumptions yielded approximately the same penetration, but the second assumption yielded significantly higher maximum moments and anchor forces. Verification of these "hand analysis" results using the CSHTSSI program showed that the top of the piling did tend to rotate into the soil and that the tip elevations from the "hand analysis" provided an adequate factor of safety. Anchor forces and moments resulting from the CSHTSSI runs were higher than those from the SMART spreadsheet analysis using either assumption concerning the soil pressures above the anchor and were thus used for design of both the concrete strut and the temporary bracing. It was determined that the penetration, anchor force, and maximum bending moment in the piling were all controlled by Case II, Surcharge Loading with Completed Construction. A summary of the results of the analysis is shown in Table 1.

Hot-rolled PZ-27 piling was originally selected for the project. After contractor and manufacturer inquiries during project advertising, the use of cold roll formed steel sheet pile was included. Since the sheet-pile details had all been prepared and dimensioned for PZ-27 sheetpiling, the contractor was required to provide revised details if alternate sheet-pile sections were proposed for use.

Welded headed studs were used to transfer uplift loads on the bottom of the concrete strut to the sheetpiling. Discussions with stud manufacturers indicated that the maximum size stud which could be welded in a horizontal position was a 3/4-in.-diam stud. The manufacturers indicated that larger studs tended to droop under their own weight immediately after welding. Using this size stud meant that studs should be spaced on 18-in. centers, or one stud per pile. Studs were located on the inside face of the sheet-pile pan.

Proposed construction sequencing for the steel sheet-pile alternate was shown in detail on the contract drawings. The weak clay layer causes instability of both the existing and excavated slopes, so excavation was carefully planned. Installation of the sheet piles with limited preexcavation is the first step. Then, excavation on the outside of the piles and partial excavation between the piles can be performed after the water has been diverted. Temporary bracing is installed, and the excavation between the piles is completed. A perforated drain pipe with risers is installed in a gravel underlayer below the concrete strut so that hydrostatic uplift pressures on the bottom of the concrete strut can be controlled by pumping until the concrete strut reaches adequate strength. The 18-in.-thick concrete slab is cast in place and allowed to cure in the dry before pumping is completed. Slope drains with flap gates are installed in the steel sheetpiling at 12-ft centers. Water can then be diverted into the low-flow channel in order to complete paving of the channel side slopes. Finally, the sheet pile is cut off at the top of the channel paving.

Precast Concrete U-Flume Design

During the design of the sheet-pile low-flow channel, there was some concern that the slag and debris could make pile driving impractical or impossible. It was decided to prepare an alternate design for a precast concrete U-flume in order to save time in the preparation of a modification if the sheet-pile flume proved unconstructible. The 180-ft-long section of sheet-pile low flow was also added to the contract for contaminated sediment removal as a test section to determine whether the sheet

Table 1
Summary of Sheetpile Analysis Results

Load Case	Analysis Type	Maximum Moment, ft-lb	Anchor Force, lb	Calculated Pile Tip Elevation, feet
Case I	Active	5,219	4,121	-3.61
	Passive	-8,477	6,495	-2.63
	CSHTSSI	-12,237	7,539	—
Case II	Active	11,794	5,341	-7.43
	Passive	8,620	7,684	-6.74
	CSHTSSI	-30,653	14,012	—
Case III	Active	4,881	3,675	-2.24
	Passive	4,858	3,769	-2.24
	CSHTSSI	13,589	5,289	—
Case IV	Active	905	625	-0.64
	Passive	-783	1,408	-0.52
	CSHTSSI	10,821	4,800	—

Notes: Active analysis type refers to analysis assuming active earth pressures above the anchor elevation. Passive analysis type refers to analysis assuming passive earth pressures above the anchor elevation. Pile tip elevation was assumed for the CSHTSSI analysis.

pile could be driven. Due to difficulties in removing the contaminated sediments and the occurrence of a flood during the performance of the contract, construction of the sheet-pile low-flow test section was delayed, and it was decided to include the U-flume alternate in the plans and specifications for the paved reach contract.

Dimensions of the precast units were determined to satisfy the requirements of ETL 1110-2-307, "Flotation Stability Criteria for Concrete Hydraulic Structures" with some modifications. Individual U-flume units were designed to be placed in approximately 5-ft lengths. Details of the precast concrete U-flume are shown in Figure 3. The design cases and their required and actual factors of safety are shown in Table 2.

This analysis required the assumption that the 10-in.-thick channel paving slab be extended over the top of the U-flume walls and that a wedge of soil angled at 20 deg from vertical above the U-flume heels be included in the analysis. If these assumptions had not been made, the base width of the units would have to be increased by 4 ft and the walls of the units would have to be 10 in. taller. In order to transport and place this section, a heavier crane would have been required, and there was concern that there could be construction difficulties with the unstable slopes.

The reinforced concrete U-flume was designed using the CASE program CUFRBC (X0095). This program is written for analysis and design of basins and channels. The program allows for the use of nonlinear soil support characteristics and analyzes the U-flume as a rigid frame. Soil responses were modeled using a wedge solution to determine active earth pressures for lateral loads and by modeling the foundation soils as elastic springs. Two different load cases were considered for the design of the precast U-flume units. The first load case was the construction with surcharge case, and the second was the extreme service case. These load cases are shown in Figure 4.

Table 2
Precast Concrete U-Flume Analysis Results

Design Case	Required Factor of Safety	Actual Factor of Safety
Construction	1.3	1.33
Normal operations	1.5	1.73
Unusual operation	1.3	1.12
Scheduled maintenance	1.3	1.30

In order to place the initial sections of the flume, it will be necessary to drive H-piles for slope stability due to the crane surcharges immediately upslope of the open excavation for

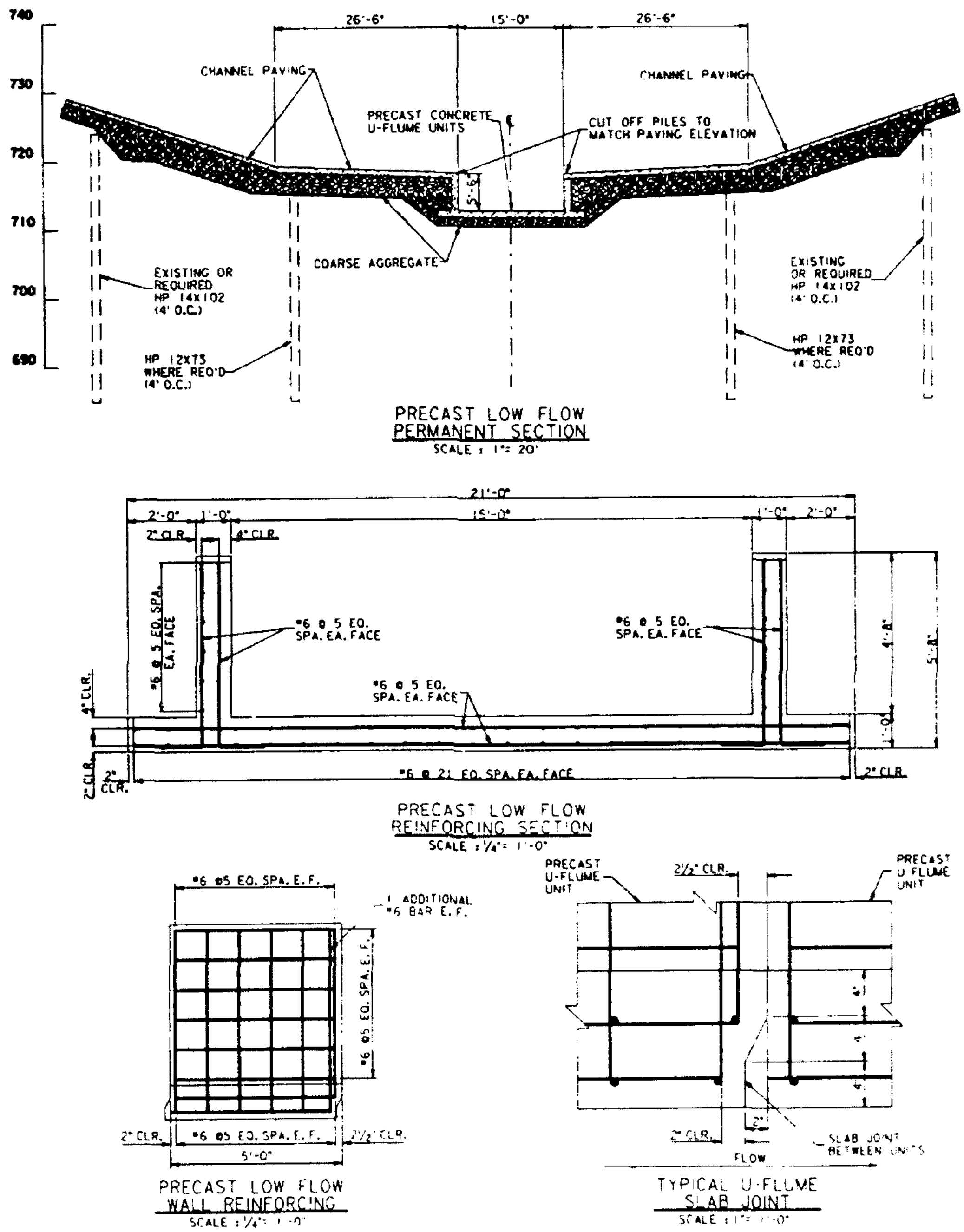

Figure 3. Precast concrete low-flow typical details

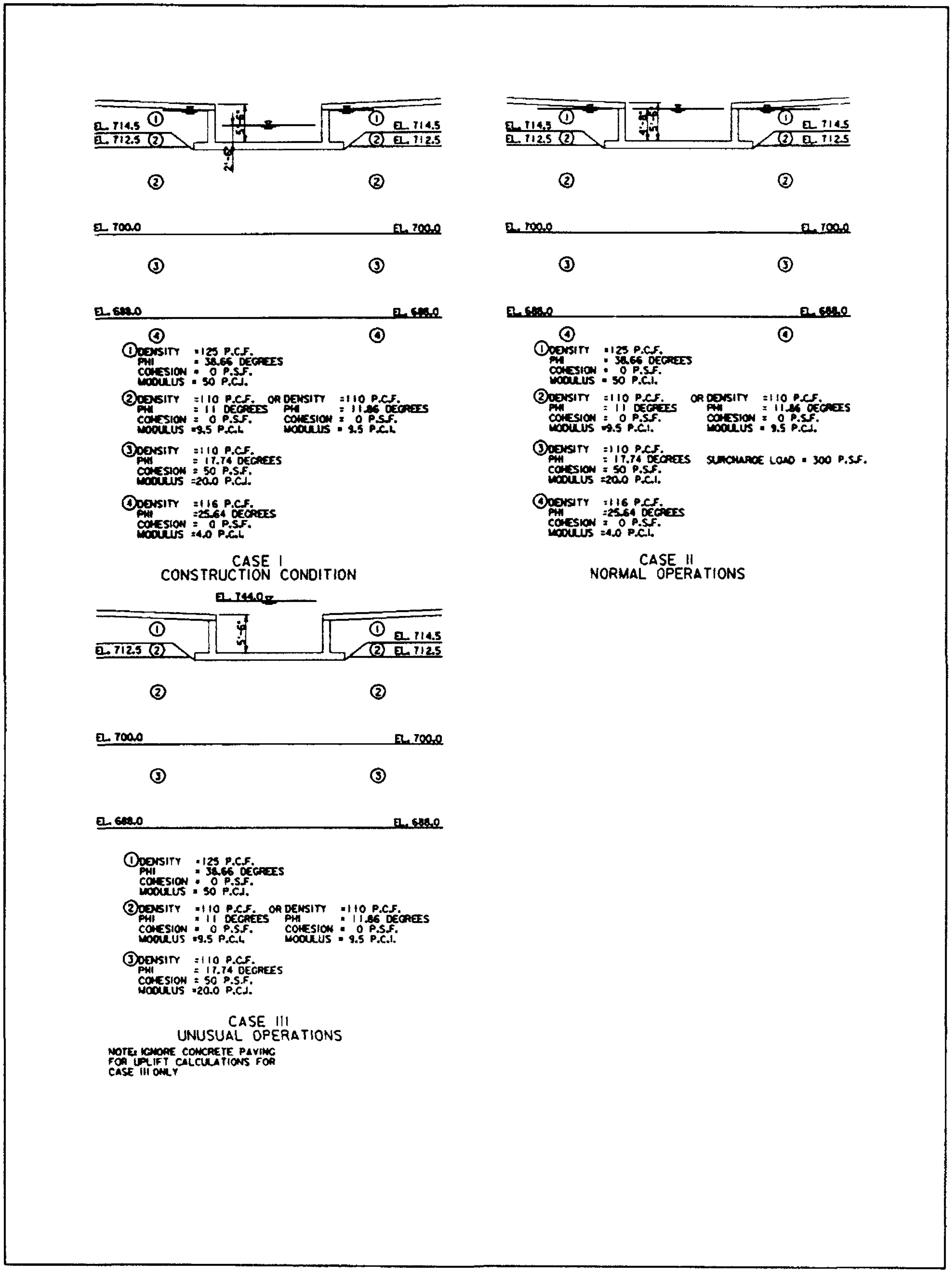

Figure 4. Precast concrete low-flow design assumptions

the flume. The H-piles were designed using the commercially available program LPILE1. Installation of the flume will be performed in the dry, with cofferdams constructed above and below the reach to be constructed. Excavation is to be performed in two phases, with excavation first performed down to the berm level at an approximate elevation of 723.0, installation of any required piles, and then the excavation for the low-flow channel is completed. A 12-in. layer of stone fill is placed, and then the U-flume unit is placed. The remainder of the excavation is filled with stone, and then the side slope paving is completed.

Relative Advantages and Disadvantages

During the design of the project, advantages and disadvantages were evaluated for each of the two low-flow alternatives. Advantages of the sheet-pile low-flow channel over the precast U-flume were anticipated to be easier control of river water during construction, reduced excavation for installation, the ability to construct the channel in longer reaches, reduced susceptibility to weather-related delays, and an estimated lower construction cost. Anticipated advantages of the precast concrete U-flume over the sheet pile included improved quality control of concrete construction, elimination of concerns about pile driving, elimination of pumping during curing of the concrete strut, and straight walls for the low-flow channel.

The sheet pile was designed to be driven to a top elevation 2-1/2 ft above the final cutoff elevation so that water can be diverted to the outside of the low-flow channel while excavation and concrete work is being performed inside the sheet-pile walls. This extra sheet-pile height provides flow capacity in the excavated area outside the sheetpiling of approximately 300 cfs which is the average flow expected during construction. The precast concrete alternative requires the use of cofferdams and diversion pipes or retention of river water upstream of the work site during working hours followed by daily flooding and cleanup of the work site.

The reduction in excavation for the steel sheet-pile alternate is due to the elimination of side slope excavation for placement of the precast units and to the use of the sheet piles as sidewalls for the low-flow excavation.

The sheet-pile low flow can be constructed in longer reaches due to the difference in river water diversion scheme. This alternative also allows driving of sheet piles ahead of the excavation and dewatering for the low-flow channel.

Since the sheet pile can be driven ahead of low-flow excavation and dewatering, work can continue during periods of inclement weather and can resume more quickly after periods of high water.

The estimated cost of construction for the sheet-pile low-flow was $6,035,000 during preparation of the GDM. Construction of the precast concrete U-flume assuming diversion of river water through pipes was estimated to cost $8,970,000, and construction of the U-flume assuming storage and release of river water was estimated to cost $11,651,000.

Government Estimate and Contractor Bids

A government estimate for the entire project was prepared for the sheet-pile low-flow alternate only, since it was anticipated to have the lowest total cost. Total project cost was estimated to be $31,852,218. There were seven bidders on the project, with all seven bids based on the precast U-flume alternate. The low bid was $20,835,073, and the high bid was $37,656,066. Four of the seven bids were less than the Government estimate. The low bidder's estimate for low-flow costs was $2,000,245, and the second low bidder's estimate for low-flow costs was $3,203,586. These bids were far less than the $6,841,085 included in the Government estimate for low-flow costs.

A comparison of the bids to the Government estimate shows two major differences in the cost of the low-flow channel. The Government

estimate for water control was \$1,017,000, while the low bidder bid \$400,000. The other difference in cost is that the Government estimate contained \$4,753,800 for steel sheetpiling and \$898,365 for the concrete in the strut, while the low bid contained just \$1,057,500 for the precast concrete sections. This shows that the selection of the precast concrete alternate was a major factor leading to the bids being considerably lower than the Government estimate.

During discussions with some of the bidders, they indicated their estimates showed that it would be approximately \$2,000,000 more expensive to construct the sheet-pile low-flow channel. Their estimates were based on placing 8 to 10 precast sections per day, while it was anticipated during the preparation of the estimate in the GDM that only two sections could be placed in 1 day. This difference in production reduced the cost of water control and also reduced construction time.

Summary and Conclusions

- Weak soils and fill consisting of slag and debris lead to the preparation of two complete designs for the low-flow channel.
- The sheet-pile low-flow channel was designed using two CASE programs and the commercial program SMART. The CASE program CSHTWAL was not able to handle the problem due to the unique geometry and soil characteristics involved.
- The precast concrete low-flow channel was designed originally as a backup design if the original sheet-pile design proved to be constructible. It was later decided to include this design as an option in the plans and specifications and to allow the contractor to decide which design proved to be the most economical and constructible.
- All of the contractors' bids were based on the precast concrete option, and those bids were considerably lower than the Government estimate. The bids all assumed placement rates for the low-flow sections that are much higher than those assumed in the GDM cost estimate.
- As of the writing of this paper, construction of the low-flow channel has not begun, so it is not possible to tell whether the contractor will be able to construct the precast concrete U-flume in the manner that he based his bid on. However, it appears that the bid relies heavily on low river levels and good weather. Should adverse conditions occur, final costs could exceed the estimate for the sheet-pile option.

London Avenue Canal Butterfly Valve Structure

by

Walter O. Baumy, Jr., PE[1]

Abstract

The US Army Engineer District, New Orleans, has developed a unique flood control structure equipped with automatic butterfly valves. The automatic feature operates on the theory of an eccentrically located vertical pin. In this particular application, the structure will be placed in a drainage outfall canal. Hurricane flood waters are prevented from entering the canal while pumped drainage is allowed to pass through the structure and exit into the lake. This paper describes the various features of the structure, basic operation of the valves, model studies conducted to develop the structure, and design aspects of the valves.

General

The London Avenue Canal butterfly valve structure is part of the "Lake Pontchartrain, Louisiana and Vicinity" hurricane protection project. This project protects the City of New Orleans from hurricane induced storm surges originating in the Gulf of Mexico. The Gulf of Mexico is directly linked to the city via Lake Pontchartrain and other small lakes and channels. Hurricane protection is provided through a series of ring levees and flood walls that ultimately tie into the Mississippi River levee system. Since the city is encircled by levees, all rainfall runoff must be pumped through the levee system and into Lake Pontchartrain or other outlets. Eighteen pump stations are located throughout eastern New Orleans for that purpose. London Avenue Canal's primary purpose is to act as an outlet for two of these pump stations, Figure 1, having a combined capacity of over 8,000 cfs. These stations are operated and maintained by the Sewerage and Water Board of New Orleans (SWBNO). London Avenue Canal is approximately 3 miles long with current protection provided by locally constructed levees and floodwalls that parallel the canal and tie into each pump station. Existing protection is generally constructed between el 7.0 and 10.0 NGVD, satisfing design criteria for a storm surge occurring on a 10- to 25-year frequency. The design storm surge for the project hurricane has a 300-year frequency. Average ground elevation within the protected area is -6.0 NGVD with some areas as low as -10.0 NGVD. Flood stage for the project design hurricane surge is elevation 14.5 NGVD which includes 3 ft of freeboard and excludes wave action.

Problem and Solutions

The project objective is twofold. First and foremost the authorized purpose of the project is to prevent tidal inundation of developed city areas via the lake/canal connection. Second, the solution must not hinder the SWBNO's ability to pump runoff into the canal during a hurricane. SWBNO also requested that any solution not reduce their pumping efficiency. The SWBNO's requirements proved to be the most challenging.

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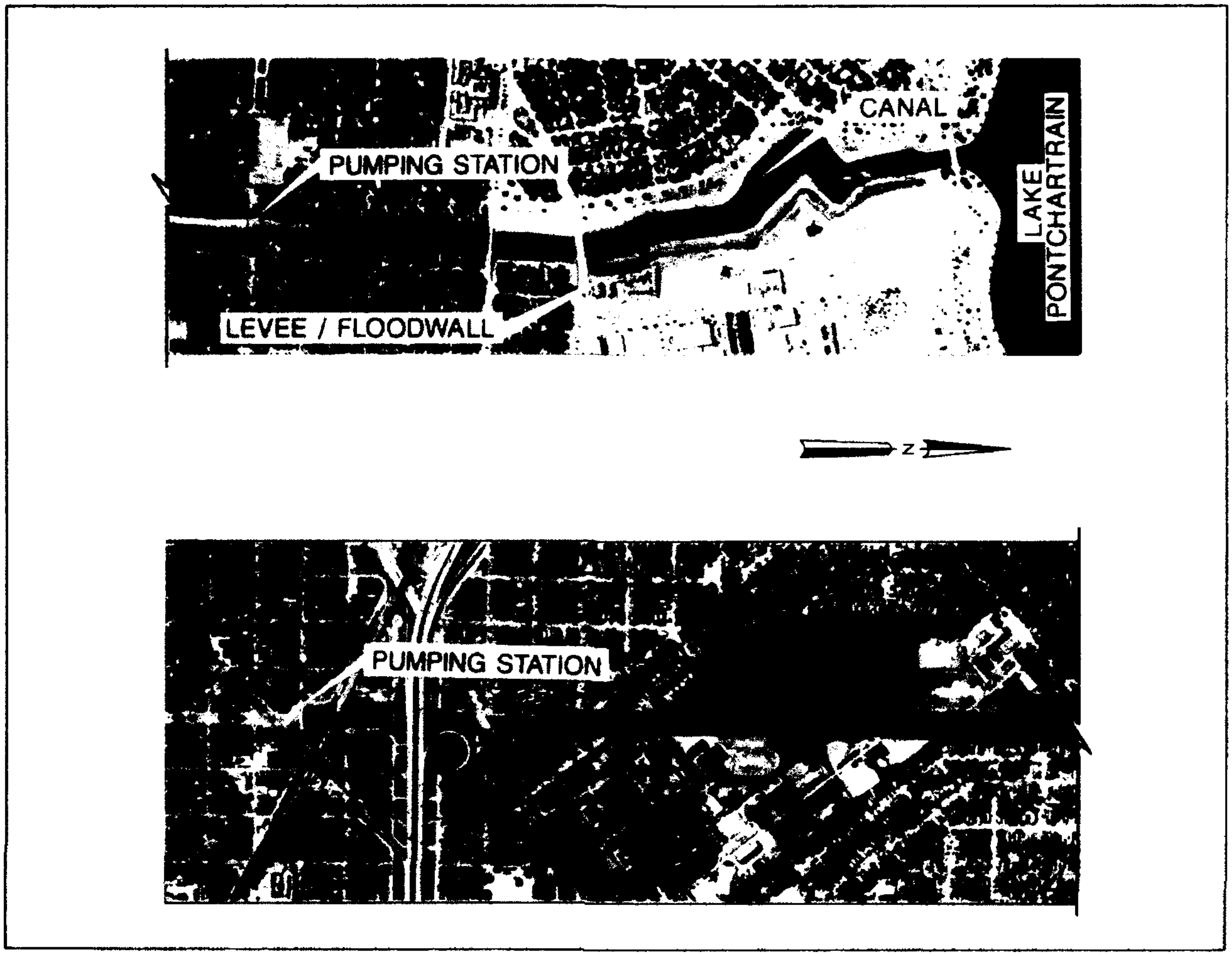

Figure 1. Plan view of London Avenue Canal

The first plan, parallel protection, envisioned raising the existing levees and floodwalls along both sides of the canal from the lake to the pump stations. Engineering problems associated with this plan were obtaining levee/floodwall section stability requirements for the required height, floodproofing or replacing nine existing bridges, constructing frontal protection at the two existing pumping stations, and working in congested areas during construction. In particular, each pump station would require partial closing to construct the frontal protection. Additionally, the nine bridges would also require closing during the modification or replacement process. This plan was estimated to cost a minimum of 45 million dollars.

The second plan considered several alternatives that provided frontal protection near the lake. These alternatives consisted of: (1) construction of closed culverts extending from the pump stations to a frontal type structure near the lake; (2) construction of a new pump station near the lake that would also serve as a frontal structure; and (3) construction of a gravity drainage structure near the lake with tainter or vertical lift gates. From these, only the gated gravity drainage structure (alternative 3) proved cost effective when compared to the parallel protection plan. In fact, it was estimated to be less than one-third the cost. Each of the other plans substantially exceeded the cost to construct parallel protection along the canal.

Operational requirements for a tainter or lift gate structure require gate closing at a predetermined time or lake stage, prior to the hurricane reaching the city. Once the gates are

closed, continued pump operation by the SWBNO could exhaust available canal storage within 20 min and overtopping or failure of the existing protection between the structure and pump stations could occur. This is a possible scenario, since the gates could remain closed for an 8- to 12-hr minimum and rainfall of 2 to 3 in. per hour can occur during a hurricane. For these reasons, the SWBNO opposed construction of this plan or any other plan requiring human judgement to determine the proper time to close and reopen the structure. The use of sensors to detect water levels and induce opening and closing of the gates was subsequently investigated and determined to be less reliable than the recommended solution.

The New Orleans District responded by developing what we call the butterfly valve structure. This is a unique structure capable of automatic response to changes in the direction of flow. Hurricane surges are prevented from entering the canal while allowing SWBNO to operate their pumps for any lake stage. SWBNO would be limited only by the ability of their pumps to operate against the increase lake stage and the confines of their levees and floodwalls along the canal. While this level is currently in the 7.0 to 8.0 NGVD range, SWBNO can incrementally increase their capabilities in the future. SWBNO intends to eventually provide freeboard above the design storm to allow continued pump operation throughout a storm.

The butterfly valve operates on the theory of a vertical, eccentrically pinned, butterfly valve (Figures 2 and 3). Each valve is locked in an open position for nonhurricane conditions to permit pumped interior drainage to enter into Lake Pontchartrain. As a hurricane approaches, each valve is placed in the active or automatic mode. The valve will remain open provided the water level in the canal exceeds that on the lakeside of the structure.

Automatic valve closure occurs when the water level on the lakeside of the structure exceeds that in the canal, i.e., whenever the hurricane surge flows into the canal. This prevents lake stages from entering the canal beyond the butterfly valve structure. The valve will automatically reopen when the storm surge receeds or pump operation causes the height of water in the canal to exceed that in the lake. Automatic operation is achieved by a combination of the valve's overall geometric shape, a 3-ft eccentricity of the vertical shaft, and placing the valve in a 12-deg offset, or trim position. This automatic operation feature allows SWBNO maximum latitude in the operation of their pumping stations during a hurricane. Machinery is provided to regulate the rate of opening and closing of the valve and also permits the automatic operation feature to be overridden. Hydraulic performance, automatic operation, and valve and machinery design were verified through model tests and analytical analyses performed by the US Army Engineer Waterways Experiment Station (USAEWES) and are subsequently discussed.

Description of Structure

Features of the butterfly valve structure, Figure 4, include eight 28-ft-wide by 43.5-ft-long reinforced concrete gate (or valve) bays, eight 16-ft-tall by 30-ft-wide structural steel valves, a reinforced concrete machinery house that traverses the structure, reinforced concrete retaining walls, reinforced concrete approach aprons, and appurtenant tie-in sheet-pile walls. All concrete structures will be pile-founded except for the approach aprons.

The valves, Figure 5, will be constructed of structural grade steel plate using all-welded construction and will weigh 44 kips each. Each valve is horizontally framed with a skin plate on each side. Vertical plates are provided for support and also to form components within the valve for floatation purposes. In the closed position, the valve seals against a bottom concrete sill extending 12 in. above the base slab, against a top concrete beam protruding 15 in. below the machinery room slab, and against the concrete pier on each side. The valve operates in a completely submerged condition for stages above el 6.0 NGVD. The valve's horizontal girders frame into a 20-in. diam pipe shaft (or trunnion) that is located 3 ft off the valve's centroidal axis (Figure 3).

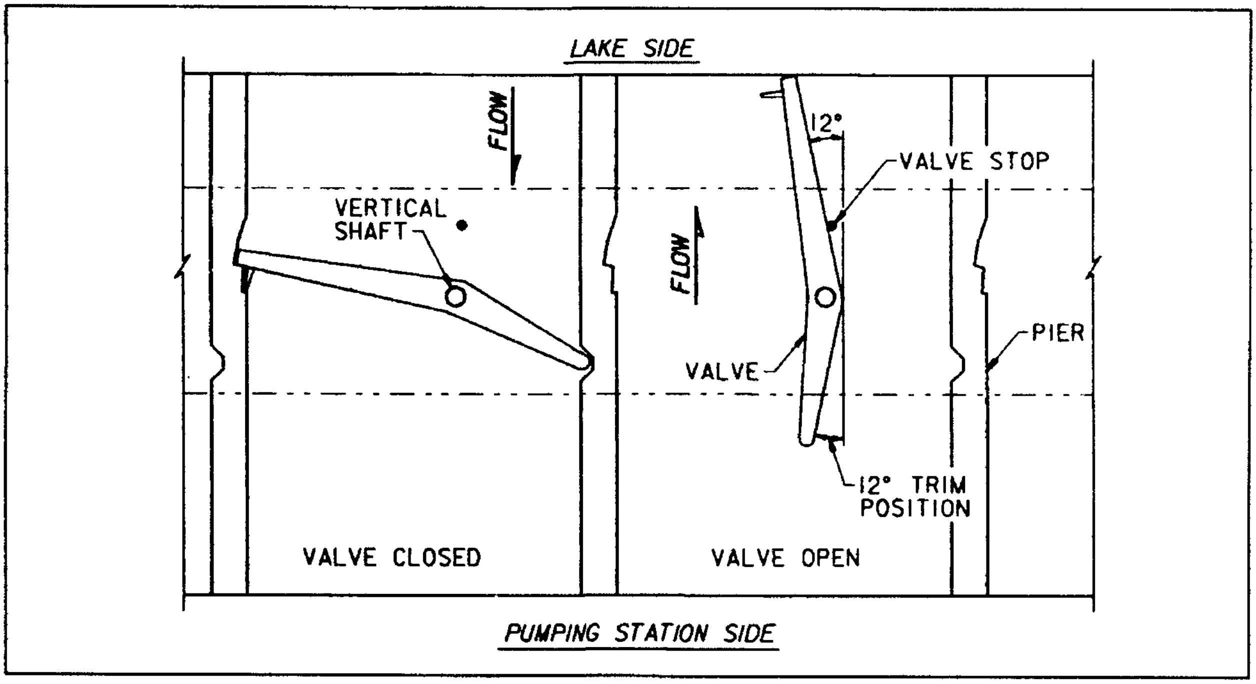

Figure 2. Plan – valve operation

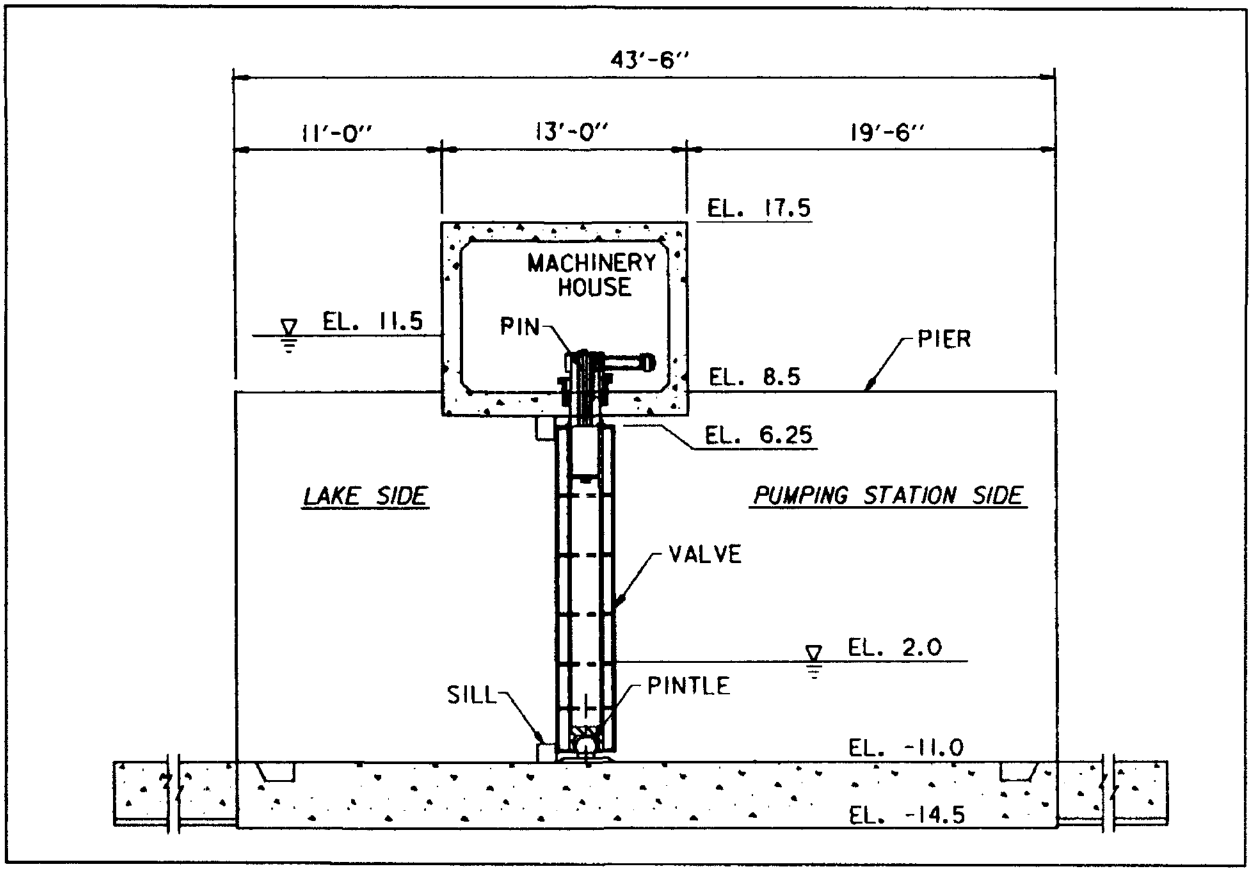

Figure 3. Section thru gatebay

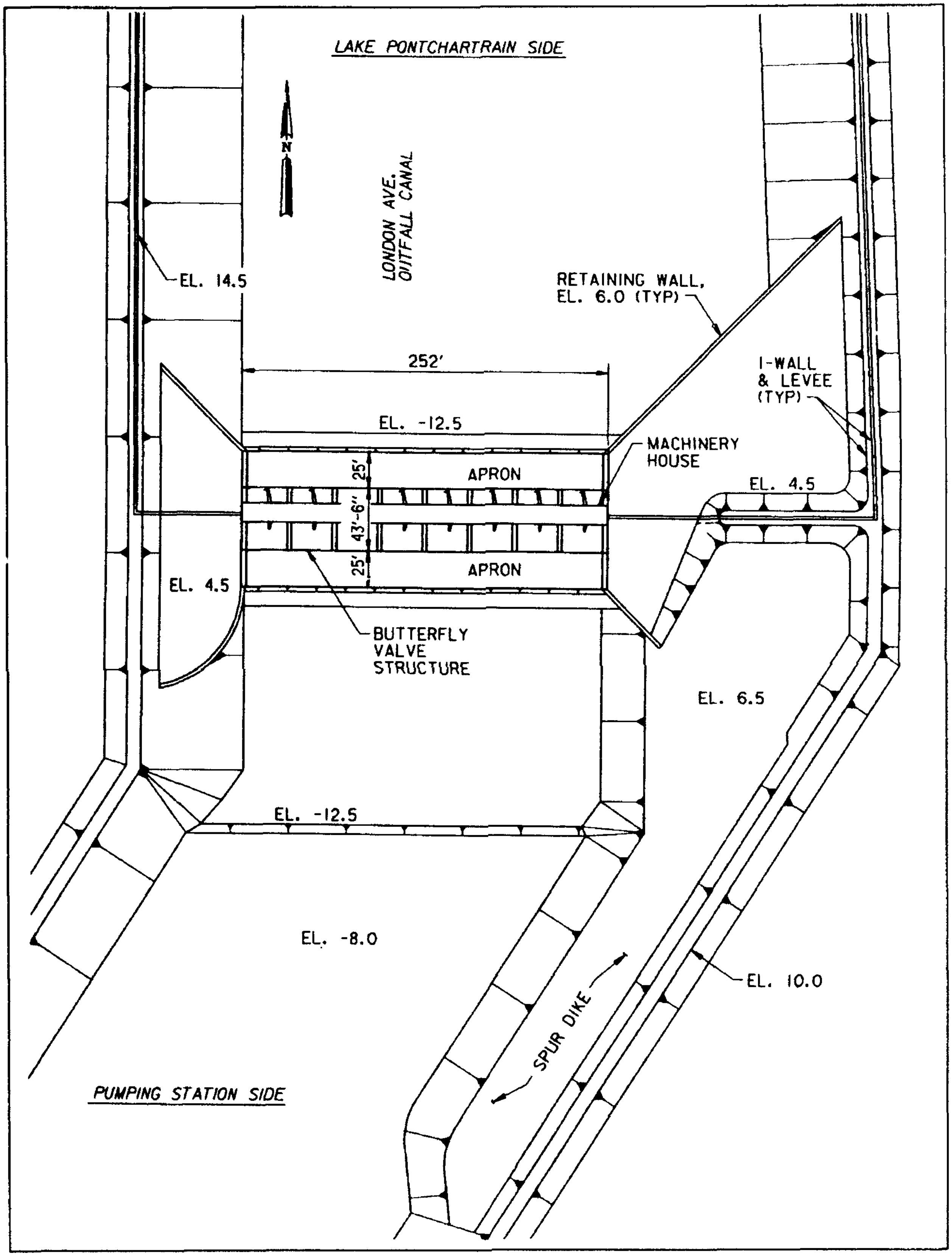

Figure 4. General plan

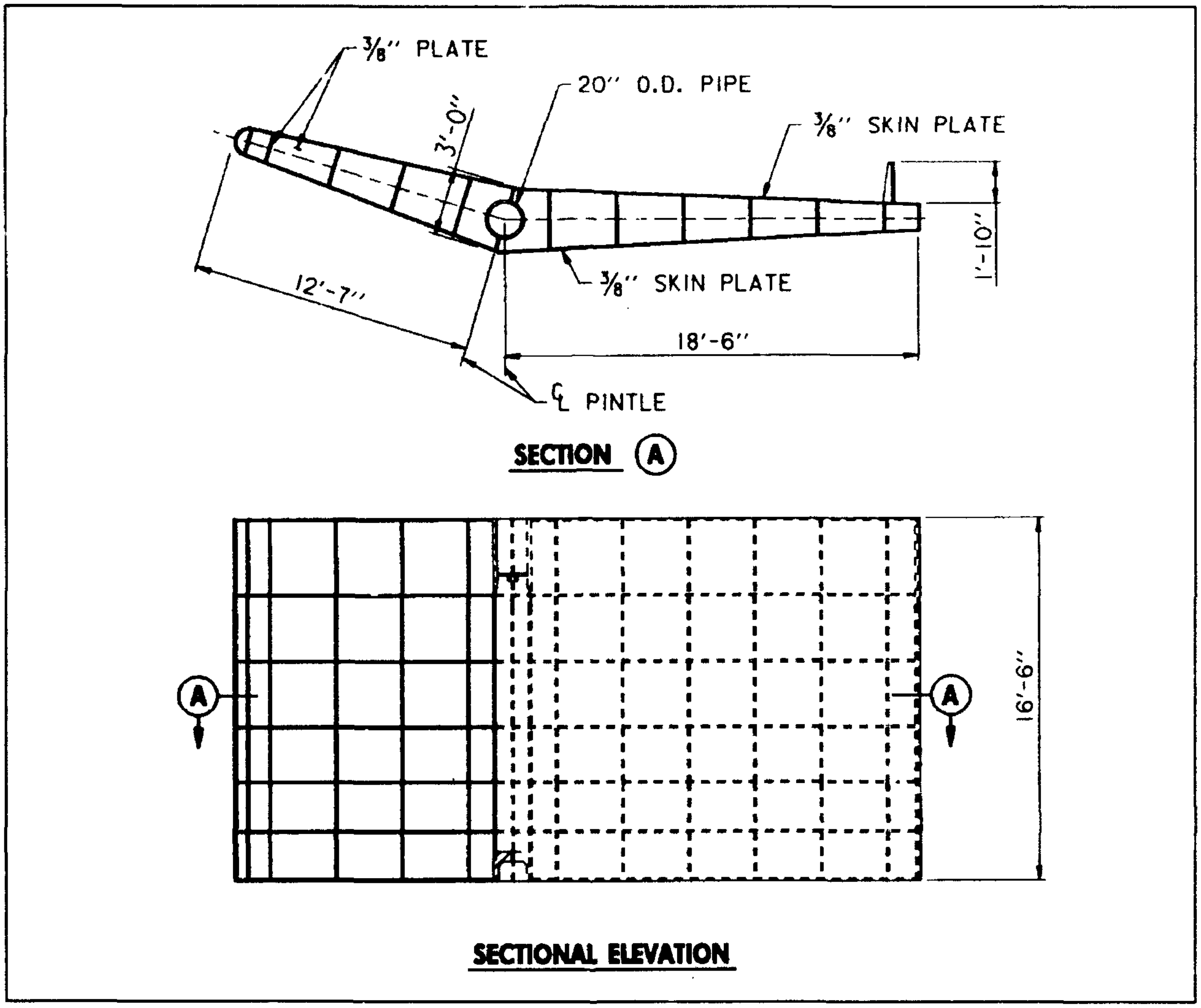

Figure 5. Butterfly valve

The trunnion rests atop an 11-in.-diam pintle and spherical bearing assembly, and the top is restrained by a 3-in.-diameter pin and bearing assembly, allowing the valve to rotate about the trunnion axis. Loads on the valve are transferred to the concrete monolith through the valve's hinge and pintle in the open positions. In the closed position, loads are also transferred into the concrete piers.

Even though valve operation is automatic, machinery will be provided for serviceability and ensured reliability. See Figure 6 for the machinery layout. Mechanical equipment is used to regulate the opening and closing speed of the valve and to permit manual operation of the valve should the need arise.

During automatic valve operation, closing and opening times are regulated to lessen impact (prevent the valve from slamming into the pier). This is accomplished through two hydraulic cylinders and a pair of parallel, adjustable flow control valves that meter hydraulic flow from the cylinders. A 7-in.-diam piston operates within each cylinder and is attached to the valve's pin by a 3-in.-diam steel rod. Each cylinder is designed for a maximum fluid pressure of 3,000 psi and a piston stroke length of 41 in. Impact of the gate closing against the pier or opening against the gate stop is diminished, since the equipment serves to dampen or cushion the blow.

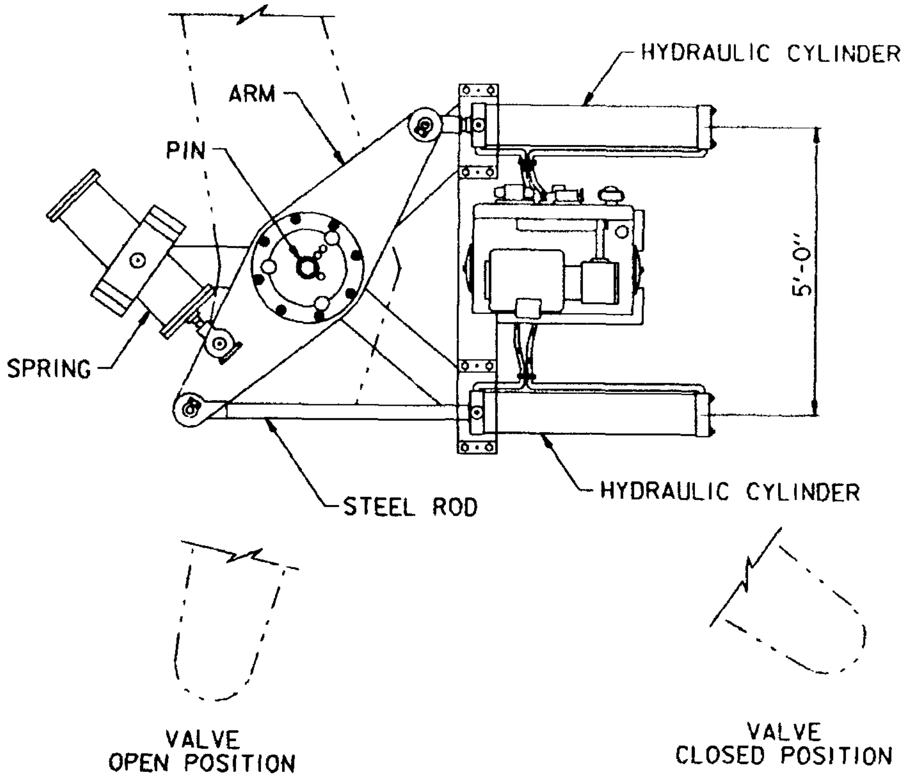

Figure 6. Machinery plan

A hydraulic unit will be provided to permit manual operation of each valve individually. The unit consists of a hydraulic pump driven by a 7.5-hp electric motor. Backup power for the motor is supplied by a diesel generator in the event of a power failure at the structure, a very probable occurrence during a hurricane. This equipment is relatively inexpensive and allows manual operation should the valve malfunction or jam during closing. The equipment is capable of generating over 500 ft-kip of torque, enough to open a valve against a 4-ft differential head, and allow flushing of debris from between the valve and sealing surfaces.

A spring is also incorporated into the machinery design and is provided to assist in the initial closing of the valve during automatic operation. The spring is connected to the valve's arm and a device anchored to the machinery room floor. The spring aids closing by supplying a preliminary closing torque of approximately 10 ft-kip to the valve in the fully open position. When flow causes the valve to begin closing, the torque applied by the spring is lessened until it reaches zero when the valve reaches a position of 30 deg from being fully open. When flow reverses, the spring will not affect the valve's operation until the valve reaches the position of 30 deg from fully opened. At this point, the spring comes into action but does not prevent the valve from fully opening as 20 to 25 ft-kip of torque is required to open the valve. That torque value will result from a flow of 500 cfs (62.5 cfs per bay) toward the lake resulting in a head differential across the structure of less than 0.5 ft.

Hydraulic Model Study

A three-dimensional (3-D) (physical) model study was conducted by the USAEWES Hydraulics Laboratory, Hydraulic Structures Division, to evaluate performance of this type

of structure since it was new to the Corps. In fact, we are not aware of any valves constructed to this scale or used for flood control purposes. The model study proved to be a necessity for this structure and site as key changes to the preliminary design resulted. The model revealed that the original structure valve did not perform as intended. Trial and error solutions in the model were necessary and resulted in the current design. A brief discussion of the model and its results follow.

The model established the optimum canal and structure alignment to permit automatic flow-induced opening and closing of the valves, established the valve geometry, and determined the magnitude of torque acting on each vertical valve shaft when subjected to various flow conditions, wave conditions, and gate openings. Wave tests were conducted by the WES Wave Dynamics Division, Coastal Engineering Research Center. The model was constructed to a 1:20 scale and consisted of the entire gated structure with eight butterfly valves, 3,000 ft of canal, 2,000 ft of lake shoreline, and 1,000 ft of lake. The entire range of design stages and flows in the canal and lake were reproduced in the model.

Uniform flow distribution through the eight structural bays was considered a necessity for the valves to open and close as desired. The model study revealed that flow through the structure was not uniform and attributed this to the structure's location and original canal design. Acceptable flow conditions were attained by modifications to the canal and structure. The canal was realigned and deepened in the vicinity of the structure. Structural modifications included reconfiguration of the structure's training walls and the addition of a rock spur dike.

Valve performance was evaluated beginning with the district's geometrical shape shown in Figure 7a. Testing indicated the Type I valve responded quickly and closed during the hurricane surge but did not completely reopen when pumping resumed. This resulted in an unacceptable head loss across the structure. Performance was improved by the addition of a spoiler, however further improvements were desired. The next phase of testing evaluated the effects of varying the eccentricity (e) between the valve's vertical shaft and centroid. The new valve opened and closed satisfactorily but still resulted in an undesirable head loss through the structure. Further improvements were sought by varying alpha(α), beta(β), eccentricity (e), and spoiler length (x) as shown in Figure 7b. The Type 33 butterfly valve shown in Figure 7c resulted and performed very well. The type 33 valve responded quickly to changes in flow direction and maintained its trim position during pumped flows. Measured head loss across the structure was 0.02 ft which was well within the 0.5 ft allowed. The model study (Leech 1987) demonstrated that the butterfly valve structure would perform satisfactorily by closing against an incoming hurricane surge, by reopening to the trim position to allow pumped interior drainage to exit into the lake, and by not causing an excessive head loss at the structure during pumping conditions.

The model also measured the torque on each vertical shaft. Measurements were made with static heads on the valves, pumped flows with the valves locked in various positions, and surge flows with the valves locked in various positions. Torque values were primarily used to design the vertical shaft and mechanical components. These items were modeled to a larger scale in the next testing phase.

Second Phase Model Study

A second hydraulic (physical) model study was performed to investigate structural behavior of the valve and to obtain data for designing the mechanical components. The model was a sectional model constructed to a 1:10 scale consisting of one full gatebay and valve, and one-half closed bay on each side. Valve model construction was of brass. Tests were conducted to measure the strain on selected valve members, to measure forces imposed on the hinge and pintle, and to measure torque on the vertical shaft. Loading conditions included static heads with the valve closed, surge flows with the valve locked in various positions, and pumped flows with the valves

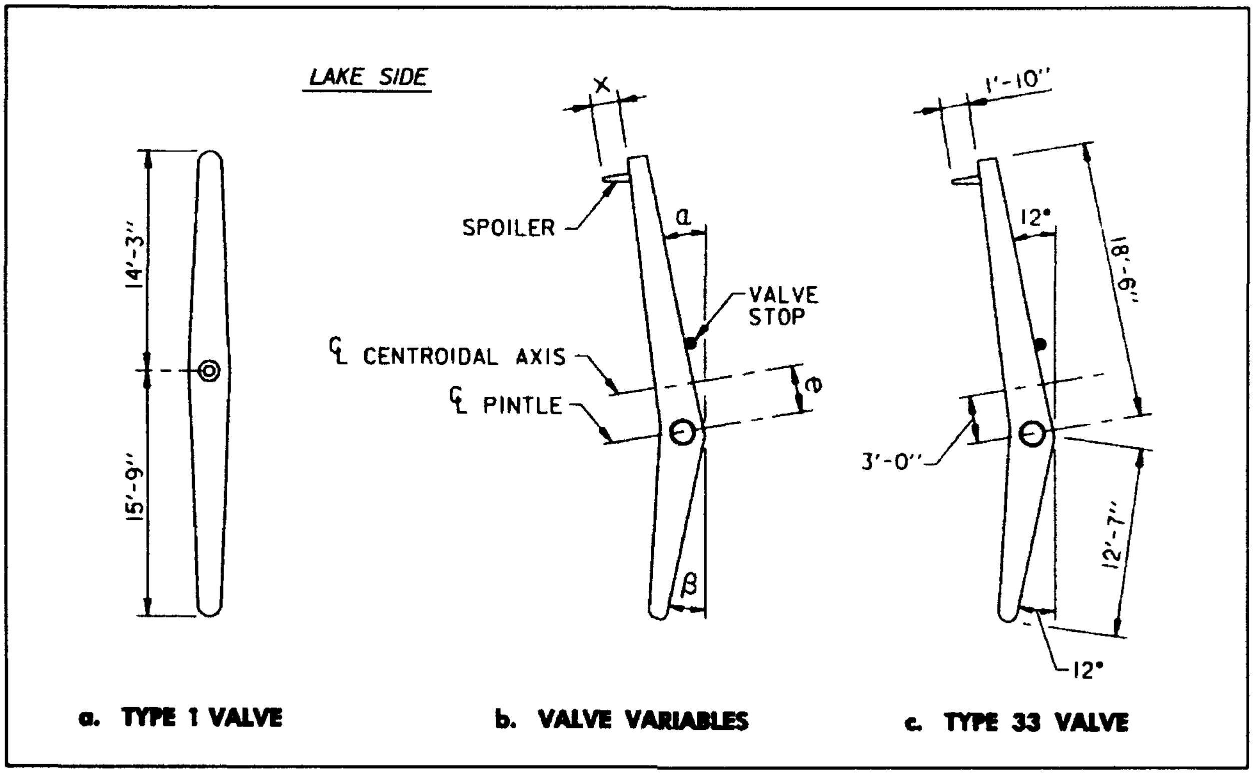

Figure 7. Model study valves

locked in various positions. Experimental strain and force measurements were made to calibrate the analytical 3-D finite element method (FEM) model.

Structural Analysis of Valve

A two-dimensional linear elastic FEM of analysis was performed by the WES Structural Mechanics Division, Structures Laboratory. The finite element method of analysis implemented in the computer program ADINA (Automatic Dynamic Incremental Nonlinear Analysis) was used. Its purpose was to verify member sizes and locations, and to select strain gage locations on the valve model. The skin plate, vertical ribs, and vertical shaft were comprised of beam elements and the horizontal ribs of shell elements as shown in Figure 8. Two support conditions were analyzed. In the first condition, the vertical shaft was pinned and free to rotate, and the long side of the valve allowed to bear against the concrete pier. The second condition also pinned the valve at the vertical shaft, but the short side of the valve was allowed to bear against the concrete pier. These support conditions attempted to analyze a worst-case condition, one where debris is lodged between the valve and concrete pier. This resulted in one leg of the valve acting as a cantilevered beam. The case of simultaneous bearing against each pier yielded stresses smaller than those previously mentioned. Bearing conditions and impact loadings will be further investigated in the 3-D analysis. The load cases listed in Table 1 were analyzed. Load Case 3 produced the highest stresses. Maximum tensile and compressive stresses occurred in the skin plate near the vertical shaft and were approximately 12 ksi.

Table 1
Load Cases

Load Case	Lake Side Elevation (ft)	Pump Side Elevation (ft)
1	11.5	2.0
2	7.0	-5.0
3	14.5	2.0

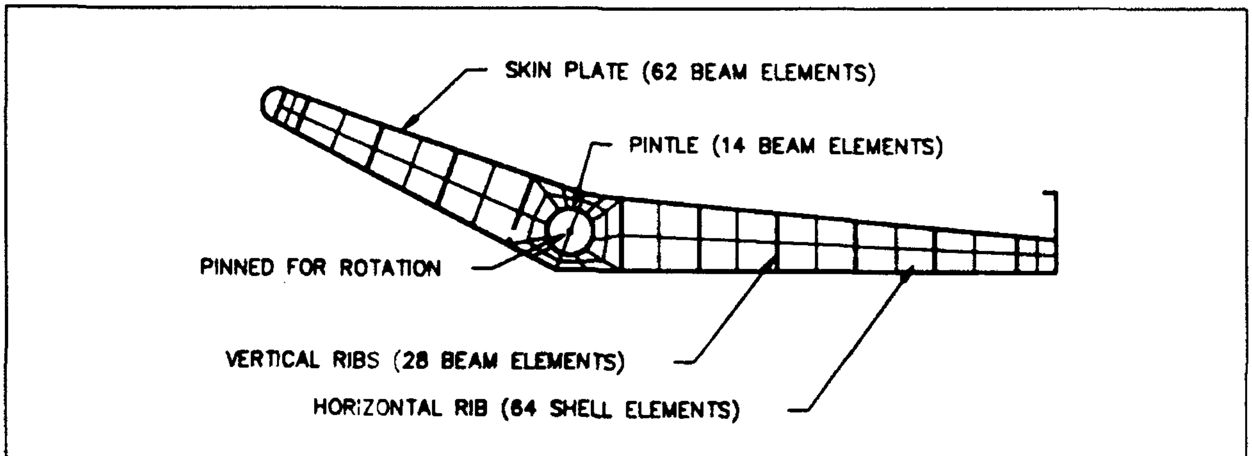

Figure 8. ADINA 2-D model

A 3-D linear elastic FEM was developed by WES (Figure 9). It was felt that a 3-D analytical model was needed to determine structural behavior of the valve for design of the model and prototype. The program ADINA was used and the model consisted of 6,490 elements. Nine-node isoparametric quadrilateral shell elements with linear elastic material properties were used. Due to the large number of nodes and elements required to construct the analytical model, grids and grid loadings were generated through computer programs specifically written for that purpose. The model was assumed to be pinned and free to rotate about the top hinge and lower pintle and supported horizontally at the piers as in the previous model. Posttest analyses showed the analytical and physically measured strains were in poor agreement. It was ultimately concluded that the experimental strain readings were inconsistent and could not be used to calibrate the FEM. Problems with the strain readings measured in the physical model were attributed to the low level of strain produced by the water loads being influenced by items that are usually minor in nature such as gauge drift and temperature. Agreement of the strains would have been a verification of the finite element grid, but was not considered a requirement. Strain comparisons between load cases also indicated a problem with consistency. Load case 3 resulted in a 30-percent increase in the valve loading as compared to load case 1. The gate is loaded such that an approximate 30-percent increase in the strain would be expected. This did not occur. In fact, the difference was sporatic; some strains increased, some decreased, and some even changed signs. The second comparison of analytical predictions and experimental data were the support forces. The support forces were in close agreement and provided one verification of the finite element grid. This verification was judged to be most important since boundary conditions are one of the biggest assumptions made. Based on the later verification, it was concluded that the finite

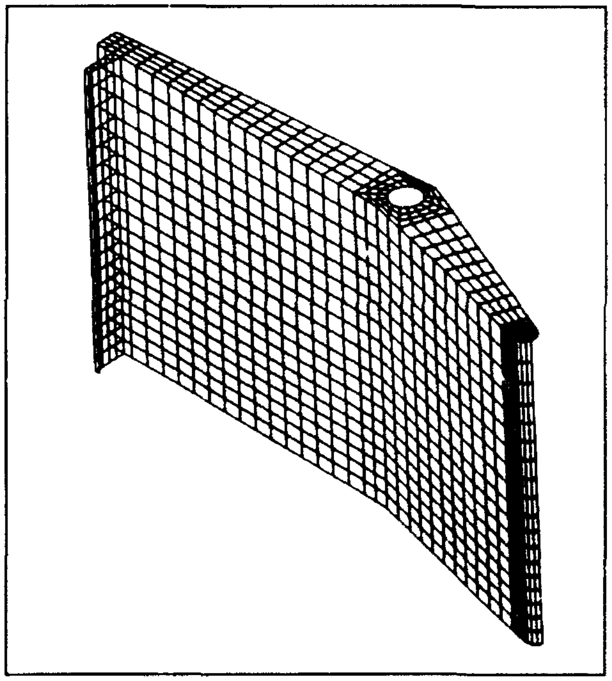

Figure 9. ADINA 3-D model

element results could be used to predict stresses in the prototype valve.

The one-tenth scale analytical model was then expanded by WES to represent the full-scale valve. Deflections and stresses on each element and reactions can be calculated to evaluate the design.

Maximum stresses for case 3 were 11 ksi in the skin plate, 13 ksi in the web of the horizontal girders, and 17 ksi in the vertical shaft. Upon completion, the FEM will be turned over to the New Orleans District and used for the final valve design.

Status

Local interests are currently seeking Federal Government funding to upgrade the existing levees/floodwalls behind the butterfly valve structure. Detail design of the structure is on hold pending this resolution.

Conclusions

The butterfly valve concept has been proven for the London Avenue Canal site through model testing. Hydraulic performance was site specific for this particular project, requiring a trial and error approach to achieve the final canal-structure configuration and the geometrical shape of the valve.

This structure type may have applications at other coastal locations.

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Miter Gate Diagonals, Stressing and Fatigue Concerns

by
Thomas J. Leicht, PE[1]

Abstract

Stressing of the miter gate diagonals on the main lock at Melvin Price Locks and Dam was accomplished in the spring of 1989. During the stressing operations a failure in the upper miter-end diagonal connection occurred. The diagonals and connections were designed in accordance with the new miter gate EM. Examination of the failure surface indicated a classic brittle failure. Analysis of the connection indicated that the stress in the failed member was in excess of 60 ksi. Moreover, additional examination and analysis indicated that fabrication procedures had created weld details that neglected fatigue reductions in allowable stresses in the connections and in the diagonal splices.

Difficulties in following stressing procedures in the field include jacking capacity, direction of jacking, monitoring diagonal elongation, nut tightening, and plumbing the gate in the dry and in the wet.

The presence of a structural engineer at the stressing operation was imperative to the successful stressing of the final configuration.

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Miter Gate Barge Impact Testing Locks And Dam 26, Mississippi River

by

Cameron Chasten[1] *and Thomas Ruf, PE*[2]

Abstract

Mississippi River Locks and Dam 26 have recently been replaced by Melvin Price Locks and Dam located immediately downstream. Prior to lock demolition, destructive testing of the vertically-framed lower miter gate in the main lock at Locks and Dam 26 was performed in November 1990. A series of barge impact loads were applied to the gate with a moving nine-barge tow. Instrumentation was installed on the barge tow and miter gate to obtain data on impact load magnitude and rate, and gate stress history and distribution during barge impact. This paper will present a summary of the testing procedures and experimental results.

Introduction

A navigation lock provides access for transporting a vessel from the upper pool to the lower pool elevation (or the reverse) through a dam. A navigation lock requires closure gates at both ends to provide a chamber in which water elevation may be varied to coincide with upper or lower pool. In order to pass a vessel downstream the chamber is filled to the upper pool elevation, the upper gates are opened and the vessel is moved into the chamber, the upper gates are closed, the chamber water level is lowered to the lower pool elevation, the lower gates are opened and the vessel is moved out of the chamber. The opposite sequence will exist for passing a vessel upstream. During this sequence, it is possible that the vessel (barge tow) will collide with the lock gates or lock walls resulting in a dynamic barge impact load.

Barge impact is an accidental event that occurs at random. It may be caused by misalignment of a barge tow in the lock chamber, excessive speed of the tow, equipment failure, pilot error, faulty communication, and many other factors. Impact loads are random in both rate of occurrence and magnitude. With the exception of the experimental work described in this paper, full-scale experimental barge impact load data are not available.

A recent survey undertaken by the Hydraulics Laboratory at the US Army Engineer Waterways Experiment Station indicated that each navigation lock experiences approximately one significant barge impact on lock gates per year (Martin and Lipinski 1990). Although barge impact is not an everyday occurrence, the damages that result can be significant. The impact may distort gate components, initiate a crack, propagate an existing crack or initial

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discontinuity, or induce fracture of a member. Distortion and cracking of members may occur locally (at the point of impact) and at locations away from the point of impact. The amount of distress at various locations depends on the load path that is assumed.

A very large percentage of the locks in the United States are equipped with double-leaf miter gates. The gate leaves each pivot about a hinged connection at the lock wall and meet in a mitered position when closed. Miter gates are framed either horizontally or vertically as shown in the schematic of Figure 1. Horizontally framed gates resist water pressure by a series of horizontal girders that are supported by vertical posts at each end. Vertically framed gates resist water pressure by a series of vertical girders supported at the top and bottom by horizontal girders. Due to the greater rigidity, comparable cost, and superior resistance to barge impact of horizontally framed miter gates, vertically framed miter gates will not be used for future construction under normal circumstances. However, vertically framed gates have been used extensively in the past, and most are still in service. In this study, vertically framed miter gates were subjected to controlled barge impact loading.

Current design loading

The current barge impact loading criteria is given in EM 1110-2-2703 (US Army Corps of Engineers 1984). For horizontally framed gates, an equivalent water load of 6 ft minimum is used for loading above the top girder, and 10 ft minimum is used below the top girder. For vertically framed gates, a 120-kip concentrated load to account for barge impact is specified. The 120-kip barge impact load is to be applied anywhere from the miter end to within 10 ft of the lock wall, allowing a 1/3 overstress. There is no documentation on the development of this loading criteria present in EM 1110-2-2703.

Full-scale test opportunity

Prior to its removal, Locks and Dam 26 consisted of a tainter gated dam and two locks; a 110- by 600-ft main lock, and a 110- by 300-ft

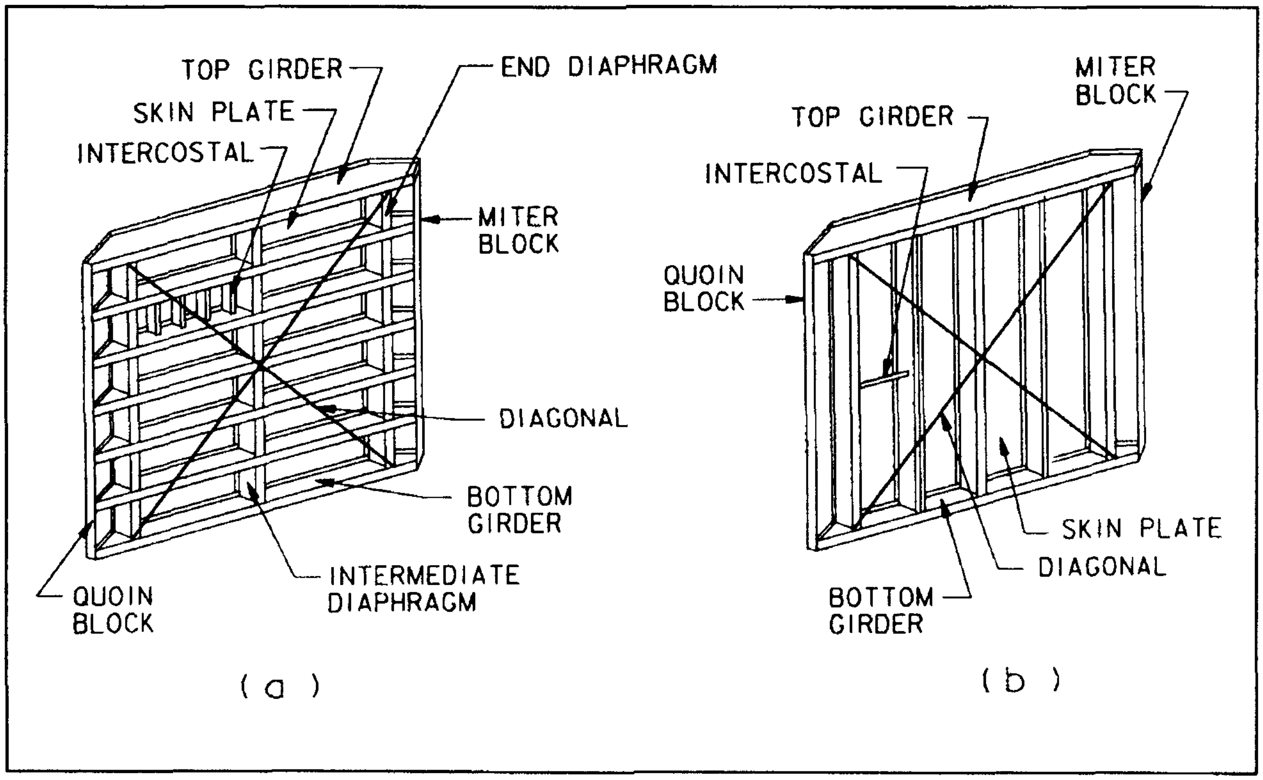

Figure 1. (a) Horizontal framing; (b) Vertical framing

auxiliary lock. Upon completion of closure at Melvin Price Locks and Dam, the replacement project downstream, and subseque't pool raise, removal of existing Locks and Dam 26 began. When enough of the existing dam was removed so that passage through the locks was no longer required, removal of the lock walls and gates began. It was at this time that the opportunity for testing became available. The lower miter gate of the main lock was tested prior to its removal. Each leaf of the lower miter gate measured approximately 45 ft high by 60 ft wide and was framed vertically with three panels. Each panel contained diagonals on both upstream and downstream faces.

Experimental purpose

The main purpose of the experimental testing was to obtain data on the magnitude and rate of a typical barge impact load, and to monitor the structural response to such a load. This information is necessary for updating design guidance which will provide meaningful barge impact design loading with experimental justification. With the development of load and resistance factor design criteria, data on load magnitude, rate of occurrence, and frequency of particular magnitude are necessary to identify a design load and corresponding load factor. Experimental data can be used for calibrating and verifying analytical models. Using verified analytical models, parametric studies with various barge mass and velocity, and structural stiffness and coefficient of restitution may be performed to extrapolate and forecast data for various circumstances. The experimental data, with analytical models may be used to establish a data base for barge impact loads.

Instrumentation

Prior to testing, instrumentation was installed on the miter gate and the barge tow. Primary information that was desired included measurement of strain in the main structural members of the miter gate leaves, and the force, velocity, and acceleration of the impacting barge tow. A total of 32 channels of data were recorded at a rate of 16 Hz using a portable digital data acquisition system.

Load monitoring system

A load monitoring system was designed to obtain the load history during impact. The system consisted of two specially designed load transducers mounted between the barge and a W14 by 398 load beam (Figure 2). Each transducer consisted of double strength 8 in. nominal diameter pipe with mounting plates welded to each end. Eight foil strain gages were mounted around the inside perimeter of each pipe to compensate for any applied bending moment, and each transducer was calibrated in a million pound universal testing machine. The load transducers were welded to the front of the barge to assure uniform load transfer through the transducer and minimize vibration between the transducers and the barge front. Small support beams were welded to the front of the barge to help support the weight of the load beam.

Strain measurement

Strain measurements were recorded using aluminum strain transducers with an effective gage length of 3 in. (Figure 3). Average strain over the gage length is monitored by measuring the electrical response of a calibrated full bridge configuration consisting of four strain gages mounted around the circular portion of the transducer. The transducers have been used effectively to monitor strains in pile driving and bridge testing. They are capable of recording strains up to 1,500 micro strain within 1 micro strain accuracy. The primary advantage of these transducers is that they may be quickly mounted to a member using small clamps or bolts. The disadvantages of the transducers include (1) sensitivity to temperature change with time, and (2) limitations on recording high frequency (greater than 10 Hz) response. The primary reason that the transducers were used in favor of conventional strain gages is the substantial savings in installation time. Under certain field conditions, it may take up to 4 hr to mount a single strain

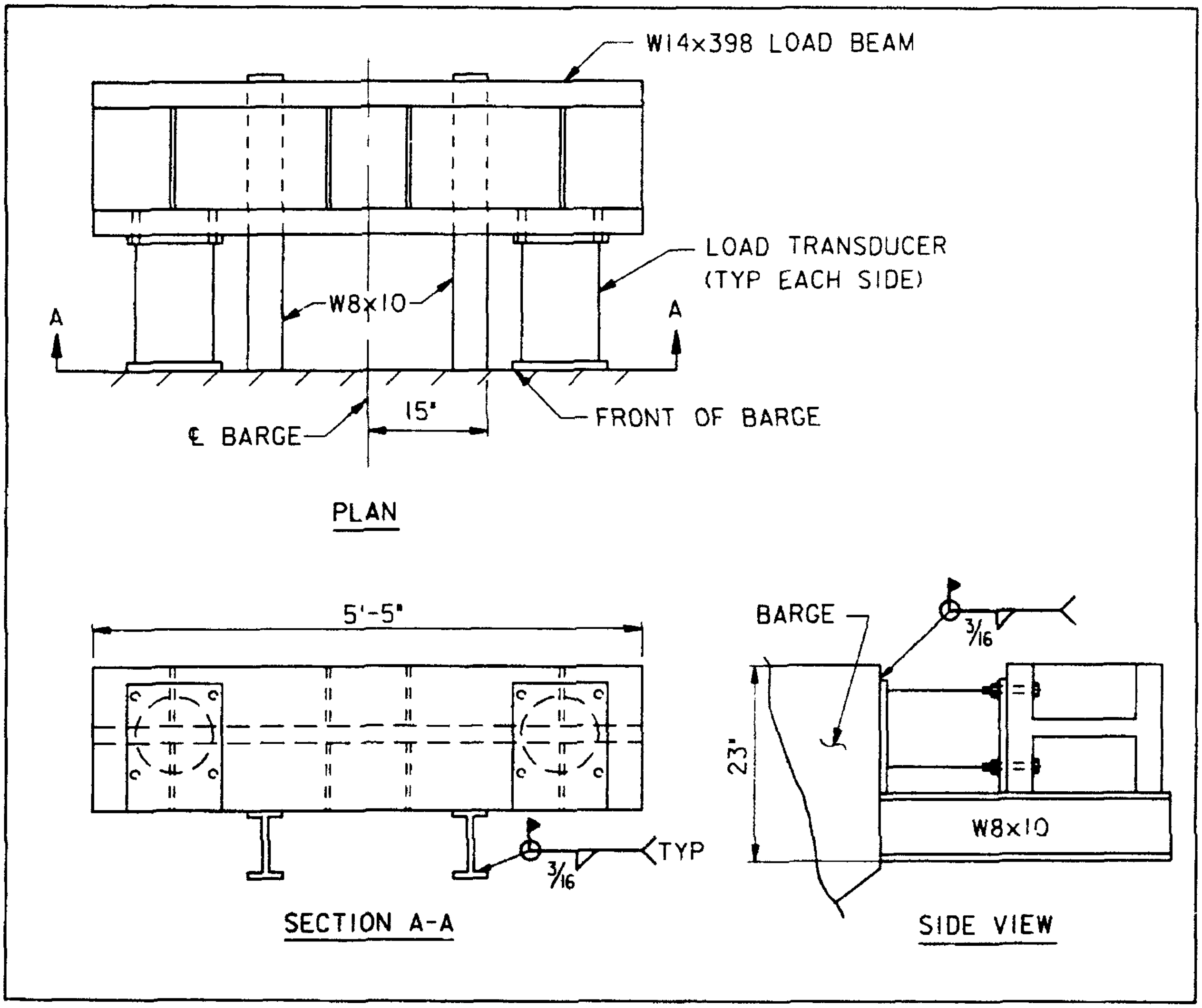

Figure 2. Load monitoring assembly

gage, but a transducer can be mounted in a few minutes.

Barge motion monitoring

Barge deceleration at impact was measured with an Entran accelerometer attached to the front of the barge tow with adhesive.

The location of the barge as a function of time was monitored using two independent systems (Figure 4). In the first system, an infrared sensor attached to the lock wall recorded signals from reflectors that were mounted to the side of the front barge at 1-ft intervals. As the barge tow moved, the reflector signal was recorded by the data acquisition system at a rate of 16 Hz. This system captured the approach velocity when the bow of the lead barge was between 10 and 20 ft from the miter gate. The second system consisted of a survey rod mounted to the top of the front-center barge and a video camera placed on the lock wall. The sight of the camera was set at right angles to the barge, and the survey rod image was recorded by video at a rate of 30 Hz during the entire event of each impact. The location of the video camera with respect to the survey rod was such that the motion of the barge was recorded throughout each impact event within a distance of approximately 12 ft of the miter gate.

Figure 3. Dynamic strain transducer

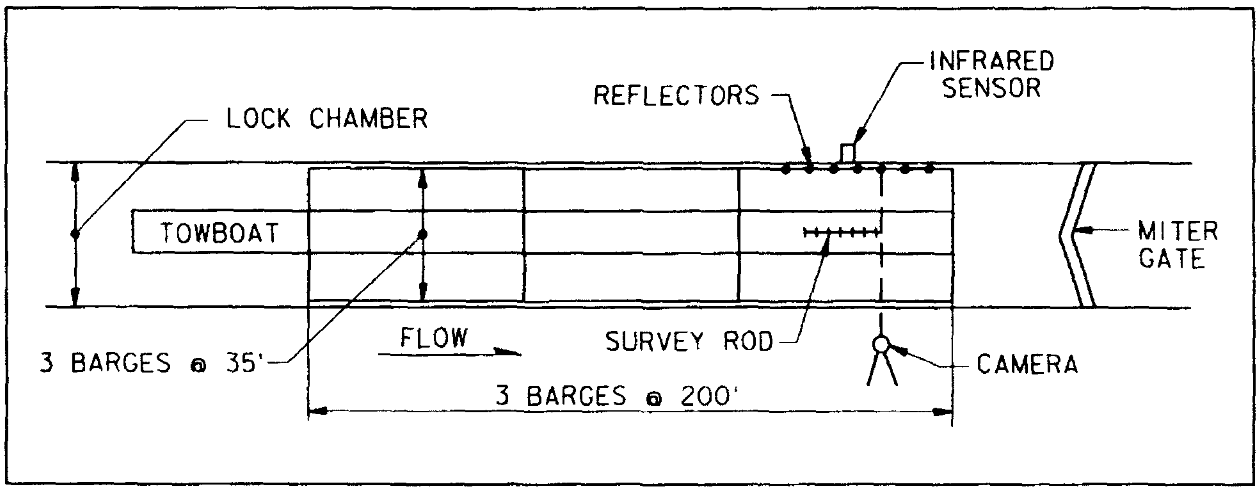

Figure 4. Velocity measurement

Test Setup

Prior to the barge impact test, the operating machinery which will be used for replacement on another project was removed from the gate leaves. In order to prepare for the barge impact test, the gate leaves had to be mitered without use of the operating machinery, the strain transducers installed on the gate, the load monitoring assembly installed on the barge, and a load-transfer beam attached to the two vertical miter girders.

At the time of the test, the pool elevation was controlled by the downstream Melvin Price Lock and Dam; therefore, there was no pool differential head on the gate. Due to identical upper and lower pool elevations, it

was necessary to push the gate leaves into a firm mitered position using towboats. The leaves were held in the mitered position with steel cables attached to the upper portion of the gate leaves and anchored on the lock wall downstream.

Figure 5 shows the location of strain transducers for the impact tests. The figure is an elevation/plan view of the land-wall leaf. The upstream and downstream elevations are essentially unfolded so the entire gate leaf may be viewed in one drawing. Conventional strain gages were installed directly below the strain transducers at locations G and H. Additional strain transducers were placed at symmetric locations on the river-wall leaf to monitor symmetry of response. The *symmetry* transducers were placed at locations opposite of points G, H, AD, AA, V, and S. These locations will be referred to as G′, H′, AD′, AA′, V′, and S′.

The donated load beam (see Figure 2) was only 5 ft 5 in. in length, and could have contacted only one miter girder, or could have been wedged between the miter girders on impact if the moving tow was off center by only a few inches. Therefore, a load-transfer beam was mounted between the miter girders to ensure that the load was transferred to both miter girders (Figure 6).

With the load-transfer beam installed, the test conditions were somewhat unique compared to an actual case due to the stiffening of the miter girder flanges. The connections between the load-transfer beam and the miter girders consisted of wedge shapes which were welded to the load-transfer beam and each miter girder flange. This ensured a solid impact and essentially stiffened the miter girder flanges. Under impact, this will only affect behavior locally; the response of the structure will not be affected significantly.

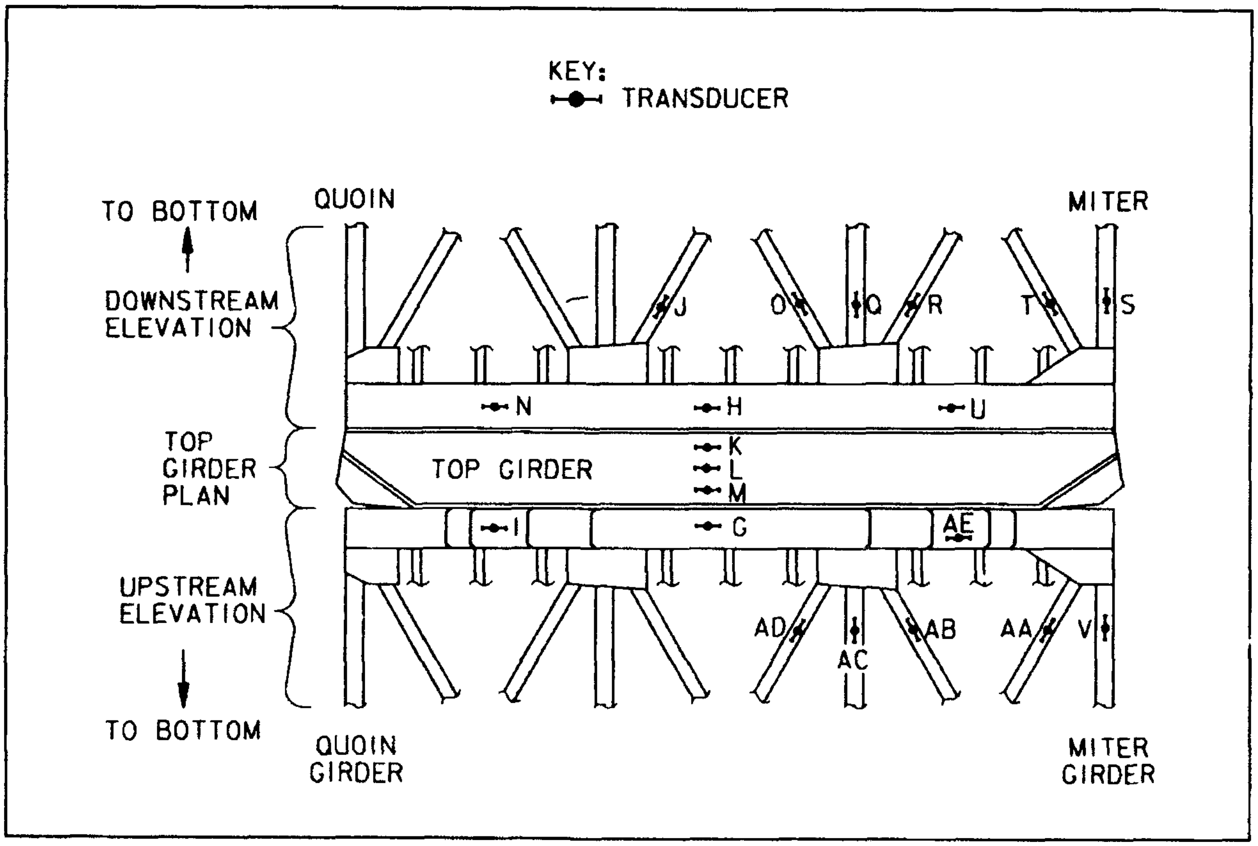

Figure 5. Strain transducer locations

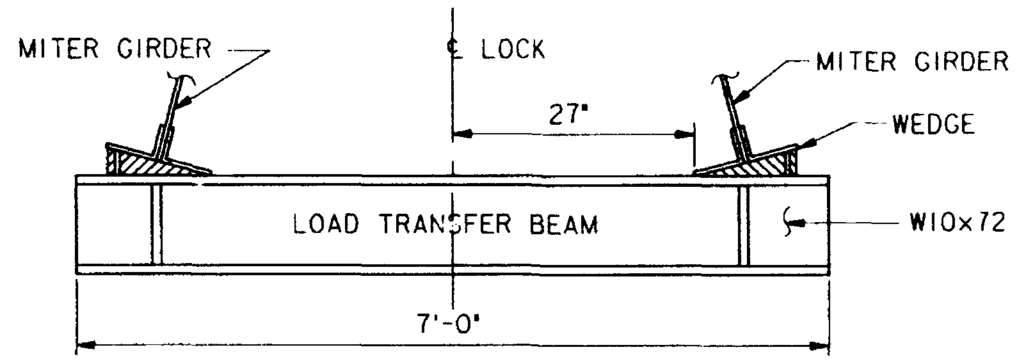

Figure 6. Load transfer beam

The vessel used to apply the impact load consisted of a nine-barge (jumbo barges) tow driven by a large towboat (Figure 4). Each jumbo barge is 35 ft wide and 195 or 200 ft long and weighs approximately 1,700 tons when loaded. A three-by-three tow configuration is 105 ft wide and 600 ft long, and was chosen since it is the largest that can fit in a standard 110 ft wide by 600 ft long lock chamber. The total weight of the vessel, consisting of the towboat and fully loaded tow, was approximately 15,950 tons or 31,900 kips. After all instrumentation was installed, the tow was pushed very slowly toward the gate, and a modest impact was applied and monitored to assure that all gages were recording.

Impact Sequence

Experimental results and discussion

A series of four progressively increasing impact loads were applied to the miter gate. For each impact, the tow was pushed downstream starting at a position where the bow of the front barge was approximately 100 ft upstream of the tested miter gate. The towboat pilot applied full engine capacity for a few seconds and then allowed the tow to coast to impact. The tow velocity was arbitrarily controlled by the length of engine operation time. The test was designed to begin with relatively low impact magnitudes to assure elastic response for at least the first impact. After each impact, the gate, barge, and load beam assembly were inspected for damage.

For the first two impacts, the gate leaves responded elastically; there was no apparent damage to the gate, barge, or load beam assembly. The third (largest magnitude) impact induced substantial local damage to the miter girders, and small cracks were identified in the welds that attached the load transducers to the barge (See Figure 2). On the fourth and final impact, the load-transducer-to-barge welds fractured and the test was stopped. The front of the barge was not damaged during any of the impacts.

On the third and fourth impacts, both miter girders experienced severe local yielding, and the webs were buckled at the point of impact as shown in Figure 7. Yielding dissipates energy, which may be desirable in some cases; the dissipation of energy due to yielding at the miter girders has the effect of protecting the rest of the structure. The damaged area was concentrated in this region; the structure did not experience any apparent damage away from the point of impact.

Strain gage results

The readings obtained from the strain transducer and corresponding strain gage located at points G and H were compared. The strain readings were essentially identical. It

Figure 7. Local damage of miter girders

is expected that under high frequency loading, that the strain transducers may not capture the peak structural response due to the large gage length and energy dissipation in the attachment. However, the low frequency vibration response (approximately 2 Hz) of the gate leaf was well within the dynamic range of the strain transducers.

Impact magnitudes

Figure 8 is a plot of the impact force as a function of time for the third impact. (Each impact resulted in similar plots.) The total time of load application ranged from 3 to 4 sec for each impact.

Table 1 summarizes results of the approach velocity, maximum recorded impact force, and peak tow deceleration for each of the four impacts. (The highly sensitive accelerometer produced acceleration records that contained considerable electrical noise data. In an attempt to remove the noise data, the acceleration histories were filtered at a frequency of 0.5 Hz.) The peak impact force was calculated using Newton's second law of motion, expressed as F = ma, as a check of the measured force. With the total mass of the impacting vessel estimated to be 31,900 kips/g, the value of ma is shown in the rightmost column of Table 1.

Table 1
Impact Force Results

Impact	Approach Velocity fps	Impact Force kips	Maximum Deceleration g	F = ma
1	0.36	442	0.0134	428
2	0.59	443	0.0132	421
3	0.94	605	0.0177	565
4	0.73	488	0.0178	568

The impact force increases with increasing velocity due to the higher *kinetic energy of* the impacting tow. The measured force for impacts 1 and 2 is nearly identical, which is not consistent with the difference in measured velocity. The results in Table 1 show the approach velocity measured by the infrared sensor. The location of the reflectors and the sensor was such that velocity was measured when the bow of the front barge was at a distance of approximately 10 to 20 ft from the lock gate. The desired velocity measurement is at the time of contact, and it is expected that a more consistent relationship will be obtained from the recorded video image of the survey rod (this data has not yet been analyzed).

For impacts 1, 2, and 3, the calculated (ma) and measured values of force are reasonably close; the calculated value of force is 3 to 7 percent lower than the measured value. The larger difference for impact 4 may be due to electrical interference in the accelerometer measurement or an inaccurate load measurement (the force transducer weld was fractured on this event). The comparison gives some confidence concerning the measured results;

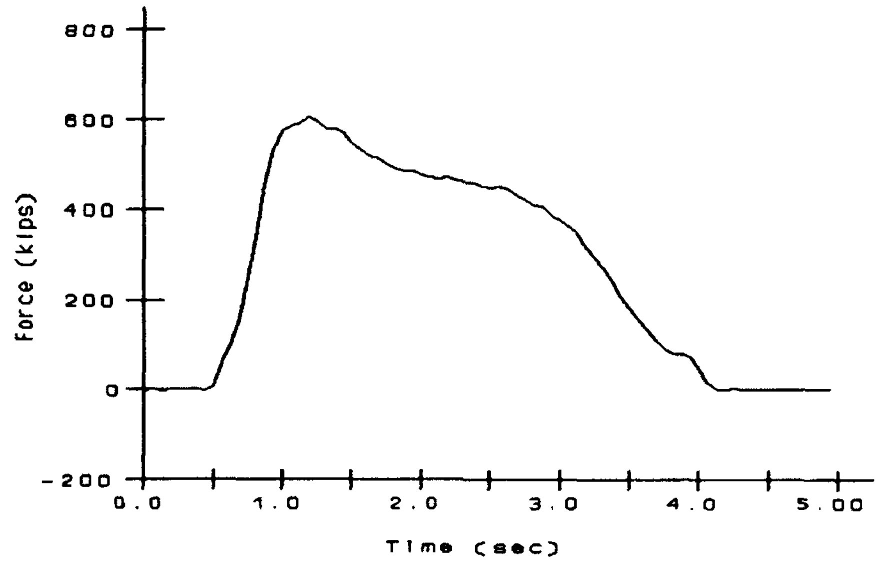

Figure 8. Impact force, impact 3

however, the measured value is considered to be more reliable since the barge mass is estimated and accelerometer records contained electrical noise interference.

Strain results and discussion

Figure 9 compares the strains at the symmetric locations G and G′, and H and H′ for

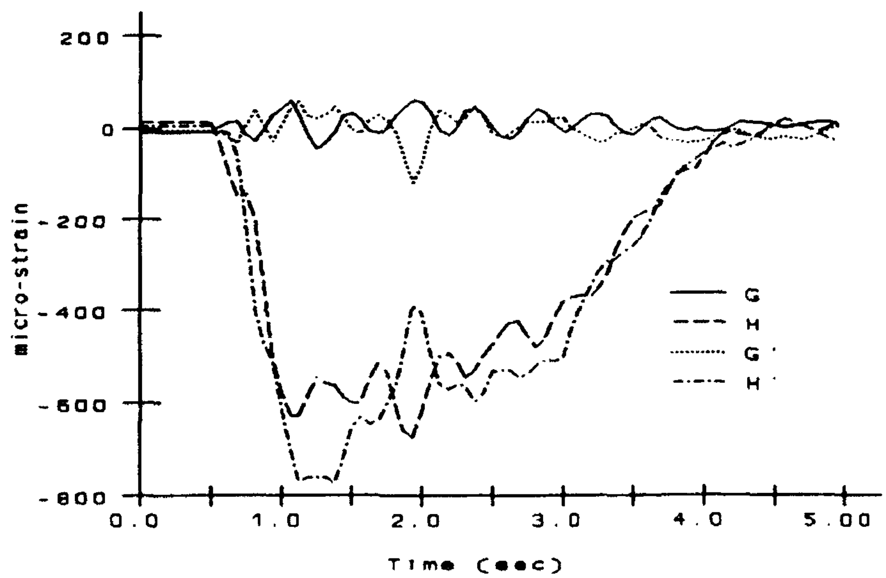

Figure 9. Symmetric strain comparison

the third impact. The nearly identical results show that the load was transferred equally to both leaves. This behavior is desirable for miter gates and the information will justify development of analytical models based on symmetry.

Strain rate

In the development of fracture control criteria, load rate and strain rate are useful quantities for establishing effects on dynamic fracture toughness. In general, the crack toughness of structural steels tested at a constant temperature decreases with increasing loading rate (Barsom and Rolfe 1987). The fracture toughness of dynamically loaded specimens (strain rate = $10\ sec^{-1}$) is much less than that of statically loaded specimens (strain rate = $10^{-5}\ sec^{-1}$).

In the development of the AASHTO fracture control plan (AASHTO 1975) the effect of load rate on fracture toughness was considered. The AASHTO criteria is based on measured loading rates for short span bridges where the maximum recorded load rate in terms of stress was about 6 ksi/sec which corresponds to a strain rate of $2(10^{-4})\ sec^{-1}$. To ensure conservatism in establishing fracture-toughness requirements, AASHTO requirements were based on test specimens using loading rates of approximately 30 ksi/sec which corresponds to a strain rate of $0.001\ sec^{-1}$ to determine the characteristic fracture toughness curve. Given data on load rate, a similar approach may be used for the development of fracture control guidance for lock gates.

Table 2 shows the maximum stress and the maximum strain rates and corresponding load rates in terms of ksi/sec for the miter girder, top girder, and diagonals. The resulting load rates are much higher than the measured in-service load rates on short span bridges (6 ksi/sec). However, they are reasonably close to the test conditions used to establish bridge fracture criteria (30 ksi/in.). These load rates are substantial compared to a static load rate (0.3 ksi/in.) and must be considered in the development of fracture control criteria for lock gates.

Table 2
Maximum Load Rates

Location	Maximum Stress ksi	Strain Rate sec^{-1}	Stress Rate ksi/sec
Miter Girder	yield	0.0019	57
Top Girder	yield	0.0024	72
Diagonals	10	0.0013	39

Stress flow

Figure 10 shows the stress flow for a particular time step (impact force = 573 kips) during the third impact. The numbers show that the largest stresses are concentrated in the top girder and miter girder near the point of impact. In the case where a pool differential exists, the large compressive stress in the downstream flange of top girder (points N, H, and U) will be reduced due to opposite hydrostatic bending strains. However, the tensile stress in the downstream flange of the miter girders will be increased with hydrostatic differential.

The diagonals on the downstream side at points R and O have an impact stress of +7.7 ksi and -9.5 ksi, respectively. These values may be significant when selecting diagonal prestress values. The total stress (impact stress plus prestress) should be greater than zero to avoid buckling of the diagonal, and less than the allowable tensile stress to avoid yielding. Regarding fracture, the impact tensile stress will add to the existing prestress and the result may be significant, especially considering the reduction in toughness due to high load rate. The large variation in diagonal forces shown in Figure 10 may be due to the fact that the prestress for each diagonal is likely not at the design level (the age of the structure is approximately 45 years).

Future work

Strain transducers were installed at each location shown by Figure 5 in less than 1 day and performed extremely well based on comparisons with the strain gages. For future experimental testing, the transducers may provide an

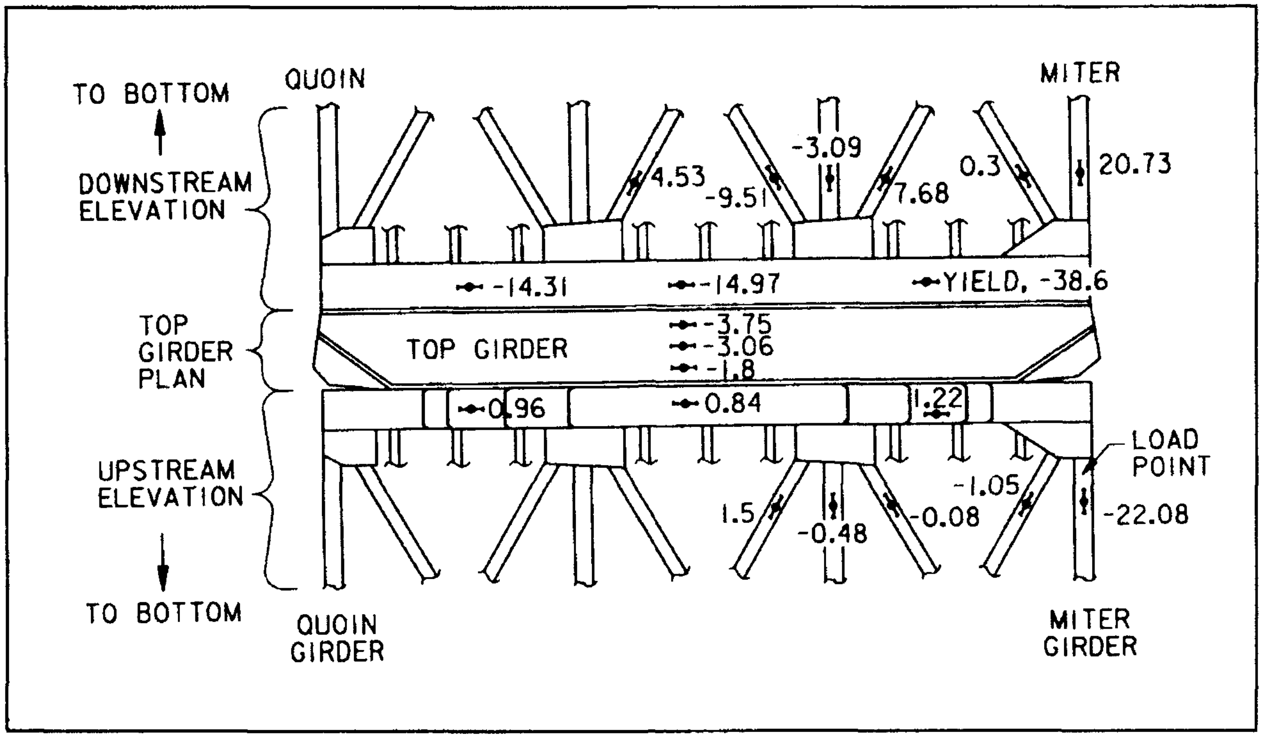

Figure 10. Stress flow (ksi), 573 kips

efficient means of monitoring the response of lock structures to various load conditions.

In the development of barge impact design loading, some assumptions and limitations on magnitude is warranted. In theory, a barge tow with a given velocity has the ability to destroy an entire gate. However, reasonable bounds on a maximum approach velocity must be assumed based on field conditions and economic design considerations. Various statistical methods must be employed to estimate the likelihood of certain events in order to determine a reasonable maximum design condition.

Analytical models

Probably the most useful result of the barge impact test is that the acquired data can be used to calibrate analytical models. Analytical models will be developed and calibrated using the experimental results. Once a reasonable analytical model is established, it can be used to forecast and extrapolate the experimental results. Various parametric studies will be performed to obtain information for variation of parameters such as barge mass, velocity, and structural resistance features such as stiffness and coefficient of restitution. Other useful information such as dynamic effects, load path, and failure mechanisms will be studied using calibrated analytical tools. With this type of information, recommended design loading (magnitude and distribution) may be established.

Horizontally framed miter gates

In general, vertically framed miter gates are relatively fragile and easily damaged when compared to horizontally framed miter gates. The vertically framed configuration is rather flexible and will likely have a lower stiffness and coefficient of restitution compared to horizontally framed configuration. Horizontally framed gates will absorb the impact in a much stiffer fashion, and under moderate impacts will most likely not be damaged. With verified analytical tools established for vertically framed gates and considering inherent structural differences, the results must be extended for study of horizontally framed gates.

Acknowledgements

This work was carried out at the Information Technology Laboratory (ITL) of the US Army Engineer Waterways Experiment Station (WES). Dr. N. Radhakrishnan is the Chief of the ITL. The project was initiated and coordinated by Mr. John Jaeger. Supporting funds were provided by the Structures Branch of the Engineering Division at Headquarters, US Army Corps of Engineers.

In order to perform this work within the limited time frame and funding, excellent coordination and cooperation were imperative. The authors express their sincere gratitude to the following for their participation and excellent cooperation: WES Structures Laboratory and Instrumentation Services Division provided strain gages and corresponding data acquisition, Bridge Diagnostics Inc. provided instrumentation and data acquisition, Hammerts Iron Works fabricated the load transfer beam, and the WES ITL photography section provided photography and video recording. Each of these participants is recognized for its excellent work and exceptional flexibility and efficiency.

The authors would like to give special thanks to the following heros and companies for their outstanding contributions. This work would not have been possible without their excellent support.

Captain Dan Brock with Ingram Barge Company donated and operated the barge tow and towboat, and provided crews with an additional towboat used in mitering the gate leaves and other miscellaneous field work.

Mr. Phil Stupp with Stupp Brothers Iron and Bridge Works Inc. donated the W14X398 load beam.

US Army Engineer District, St. Louis assisted in pretest coordination, provided miter gate drawings, and contributed necessary equipment and personnel required for installation of instrumentation, removal of material coupons, load beam fabrication and delivery, and miscellaneous field work.

Mr. Walter Giffhorn and Mr. Jack Luhr with Luhr Bros. Inc. donated towboat and crane with necessary crews to assist in various field activities including gate mitering and maneuvering material.

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Miter Gate Sill Repair Caisson

by
Ralph B. Snowberger, PE[1]

Abstract

During the low water season of 1988, extensive damage occurred to the lower miter gate sill of the 1200-foot lock of Locks and Dam 52.

To facilitate sill repair, ORL designers designed a caisson structure which could be easily handled, set in place by the LRS derrick crane, be watertight, and have room to work in.

The caisson was built in two sections which were assembled in the field after the initial placement over the sill. Access space and work space were made available by the reduced amount of X-bracing used.

The caisson was placed in one day. The damaged concrete was removed and as soon as the area was cleared, the iron work was ready. The entire operation was completed three days ahead of schedule.

Locks and Dam 52 and Locks and Dam 53 are slated for replacement with the Olmsted Locks and Dam. The original structures were completed in 1929, each with Chanion Wickets at the navigable pass and a 600-foot by 100-foot lock. River traffic became so heavy during the 1960's that we planned and built "temporary" 1200-foot by 110-foot locks at Lock and Dam 52 and later at Lock and Dam 53. These new locks have walls made of gravel-filled sheet pile cofferdam cells and concrete monoliths at each end that contain the miter gates and the culvert valves for filling and emptying. Because they are temporary structures, there are no provisions for maintenance dewatering; therefore, when a problem did arise, we needed to have some special construction.

The problem that arose was damage to the sill at lower miter gates. Our miter gates on our high lift locks are horizontally framed, e.g., the water load is carried along ribs from the miter (the meeting point of the two leaves) to the quoin (the hinge of the leaves at each side of the lock); therefore, leaving no water load against the sill from the gates. The miter gates at Locks 52 and 53, however, are low head (12-foot ±) gates and are vertically supported, e.g. the water pressure is resisted by ribs that bear against the sill at the bottom and against a horizontal "spandrel" beam at the top (the spandrel beam acts in the same manner as the horizontal ribs in the high gates—against each other at the miter gate and against the lock wall at the quoin). Because of this, having a sound sill is critical.

The sill was first damaged during the low water period in 1984 when a tow left the lock. When a tow leaves a lock, there's a tendency for it to "squat" down into the water. If the tow is loaded to the full 9-foot draft and moves

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at a rate of 0.2 feet per second, water must be replaced in the lock at a rate of 190 cfs. Additionally, the propeller draws water away from the tow, thus increasing the squat as the stern end barges cross the sill.

The repairs to the 1984 damage were accomplished by setting a small caisson over the top and upstream face of the sill and pumping it out, then repairing concrete and replacing the damaged armored edge. Because of the size of the caisson and the extent of the damage, several settings of the caisson were required. The repairs were limited to restoring the sill to the "as-built" condition - no upgrading - construction for the Olmsted Project would be well under way in five or six years!

The river level got low and the sill was more extensively damaged during the 1988 drought. The sill was inspected first by divers and then, in the more serious areas, a quick fix was accomplished using the 1985 caisson. Damage had extended to the downstream face, so it was decided that a new caisson should be made that would enable us to rebuild the entire sill.

All unsound concrete and steel had to be removed and a steel grid framework was anchored in place and filled with remedial concrete. No. 8 "U" reinforcing bars were grouted into the sill. In addition, No. 8 anchor bars to hold the steel framework were grouted into the sill. One large open bottom caisson was built to straddle the sill so the entire sill could be dewatered and repaired. Without multiple set-ups, the caisson could be set when river conditions were such that the navigable pass was open, but the water level was not forecasted to be above the top of lockwalls.

The caisson structure had to be easily handled, set in place by the LRS derrick crane, be watertight, have room to work in, and yet support a water load of approximately 30 feet. With the use of computers and Corps software, we were able to quickly study various design alternatives and loading conditions, and to choose a structural system which led to an efficient, timely, and effective repair.

The caisson weighs less than 100 tons so it could be handled by LRS and was built in two sections. The sections were assembled in the field after the initial placement over the sill. The caisson consisted of steel construction. The water loads are primarily resisted by three horizontal frames, one above the other. Skin plates and wide flanged beams transfer the loads to the main members. The three frames have a combination of X-bracing and moment connections. Access and work space were made available by the reduced amount to X-bracing used.

The requirement to be watertight at the joints was met by the use of sealing face plates. As previously mentioned, computer programs added greatly in determining forces and reactions in the boomerang-shaped structure.

Three days had been scheduled for placement of the caisson. It was placed in one day. The downstream surface was modified to meet the contour of the paving blocks, sealed off, and the caisson pumped out (see photos of caisson). That allowed the sill work to begin ahead of schedule.

The damaged concrete was removed using a cable saw. As soon as an area was cleared, the iron work was begun and then concrete placed as quickly as the steel was ready. The access space provided in the caisson design greatly aided the construction. The entire operation was completed in fifteen days, three ahead of schedule, but only one before a flood. Work began on 3 April 1990 placing the caisson, and it was removed on 17 April 1990. The water rose above the top of lockwalls on 18 April 1990.

Material cost for the caisson was $122,512. It was built at LRS for a labor cost of $267,790, a total of $390,302. The cost of the sill repair was estimated to be $300,000. Ease of installation, available access space, and a minimum amount of leakage contributed to a timely repair.

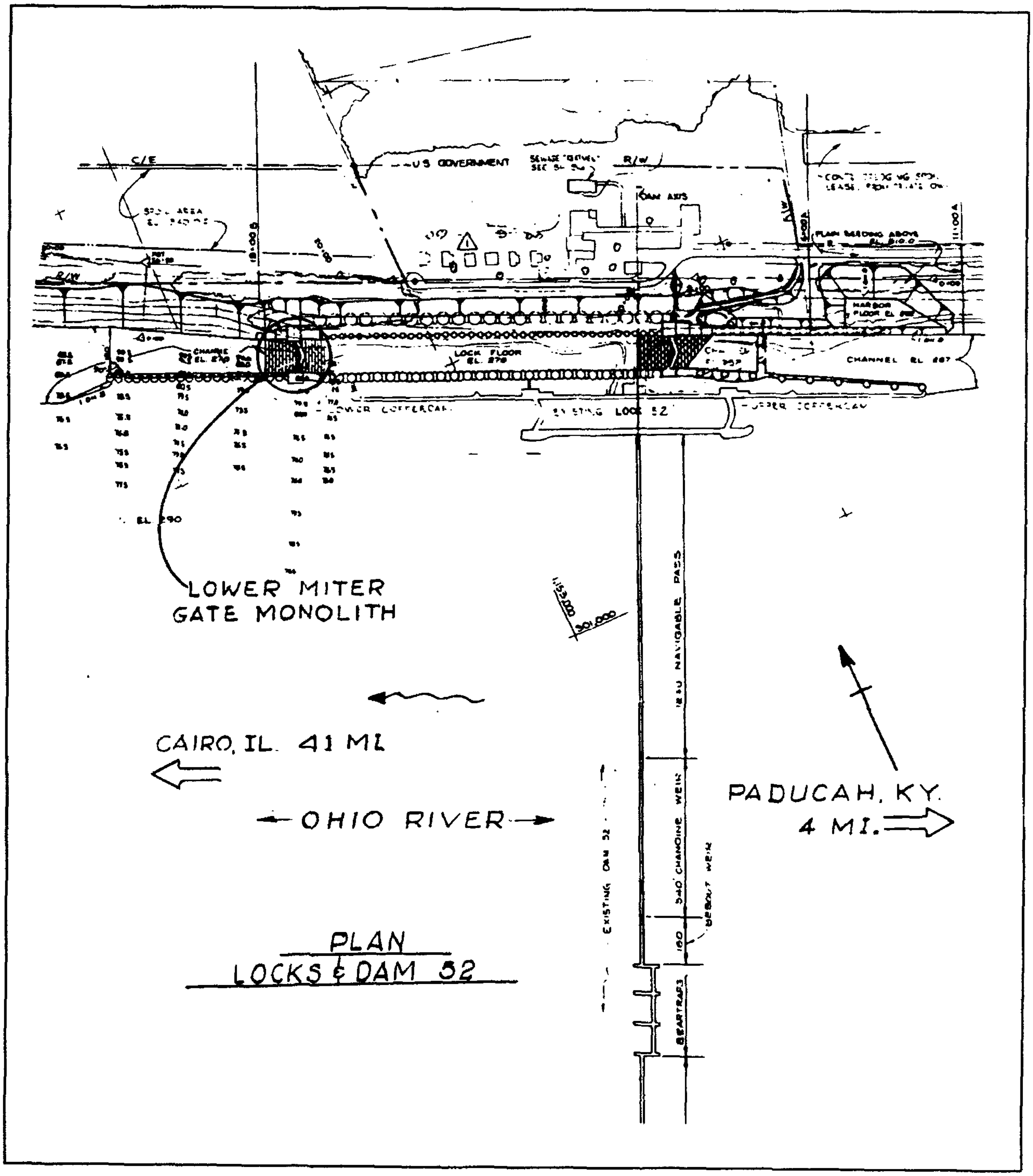

Figure 1. Plan, Locks and Dam 52

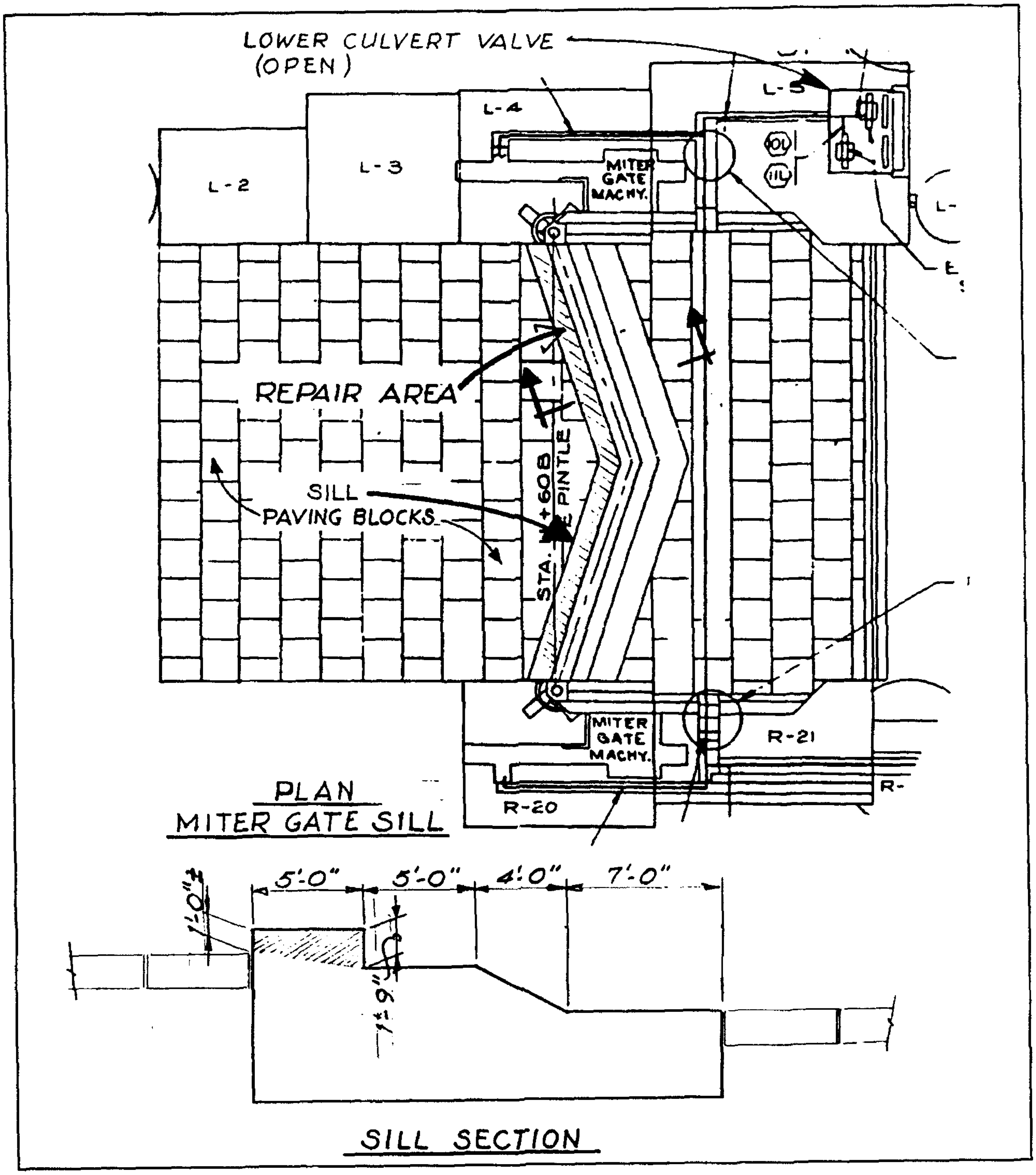

Figure 2. Sill section

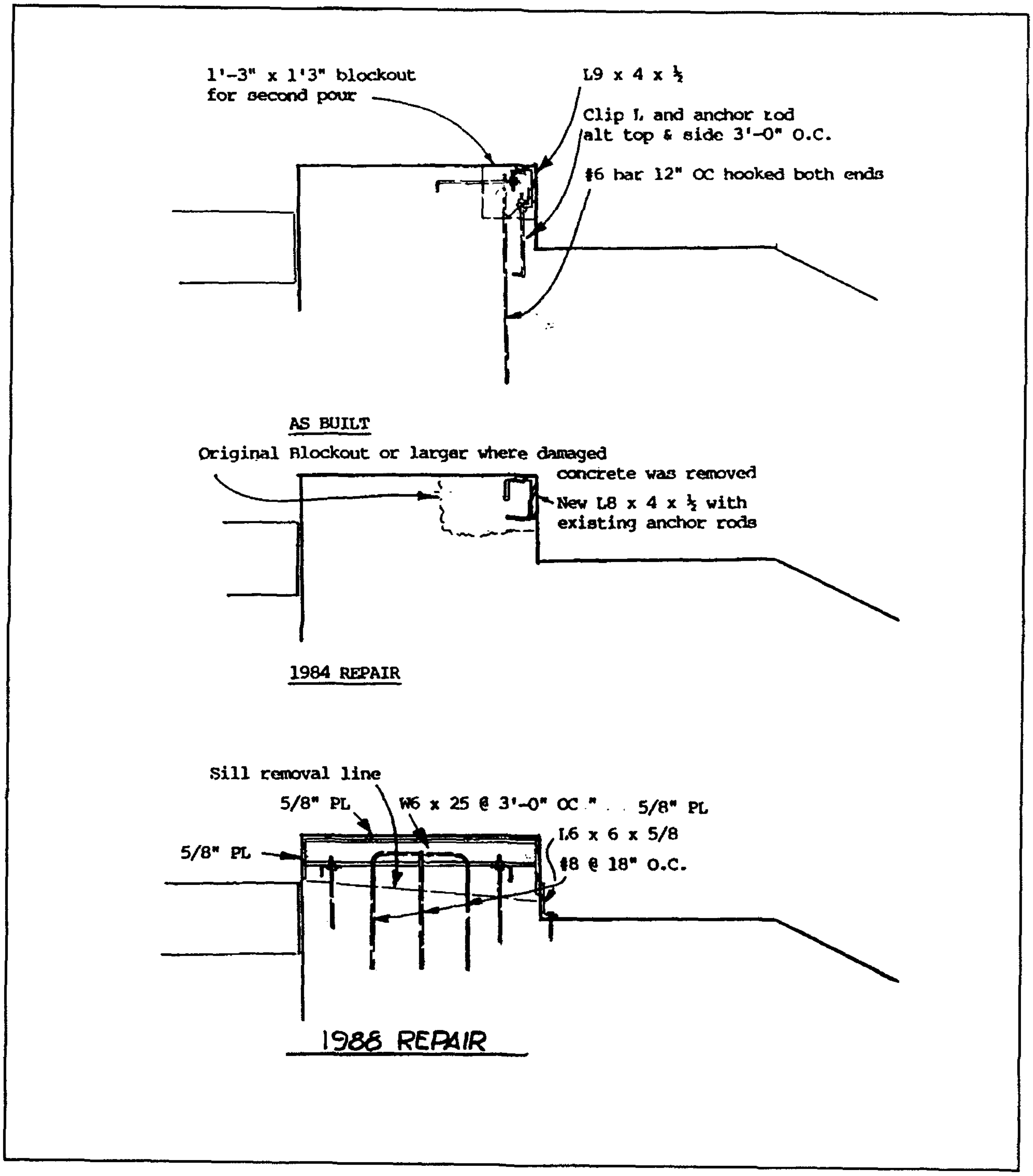

Figure 3. 1988 repairs

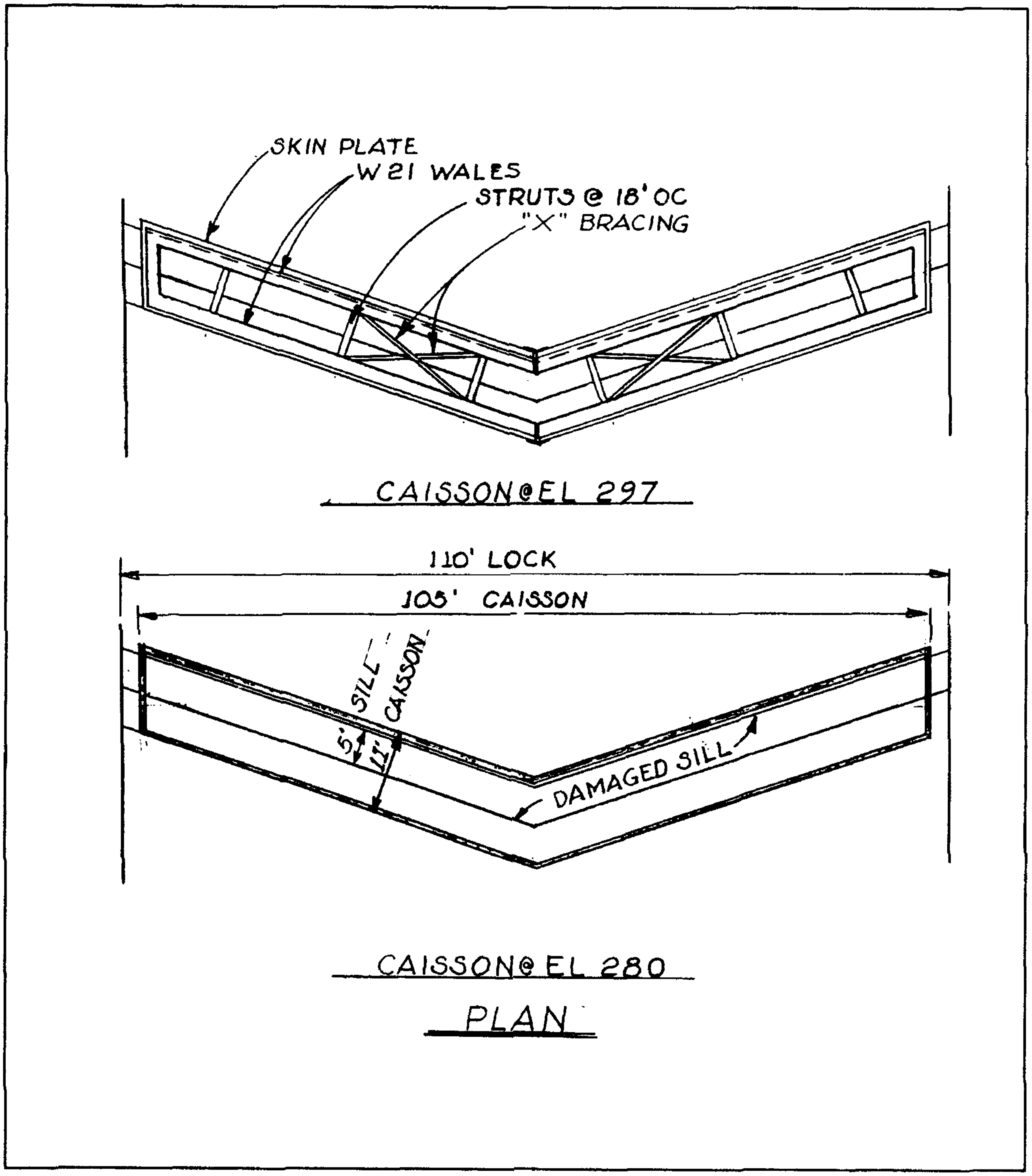

Figure 4. Caisson at El 297 and El 280

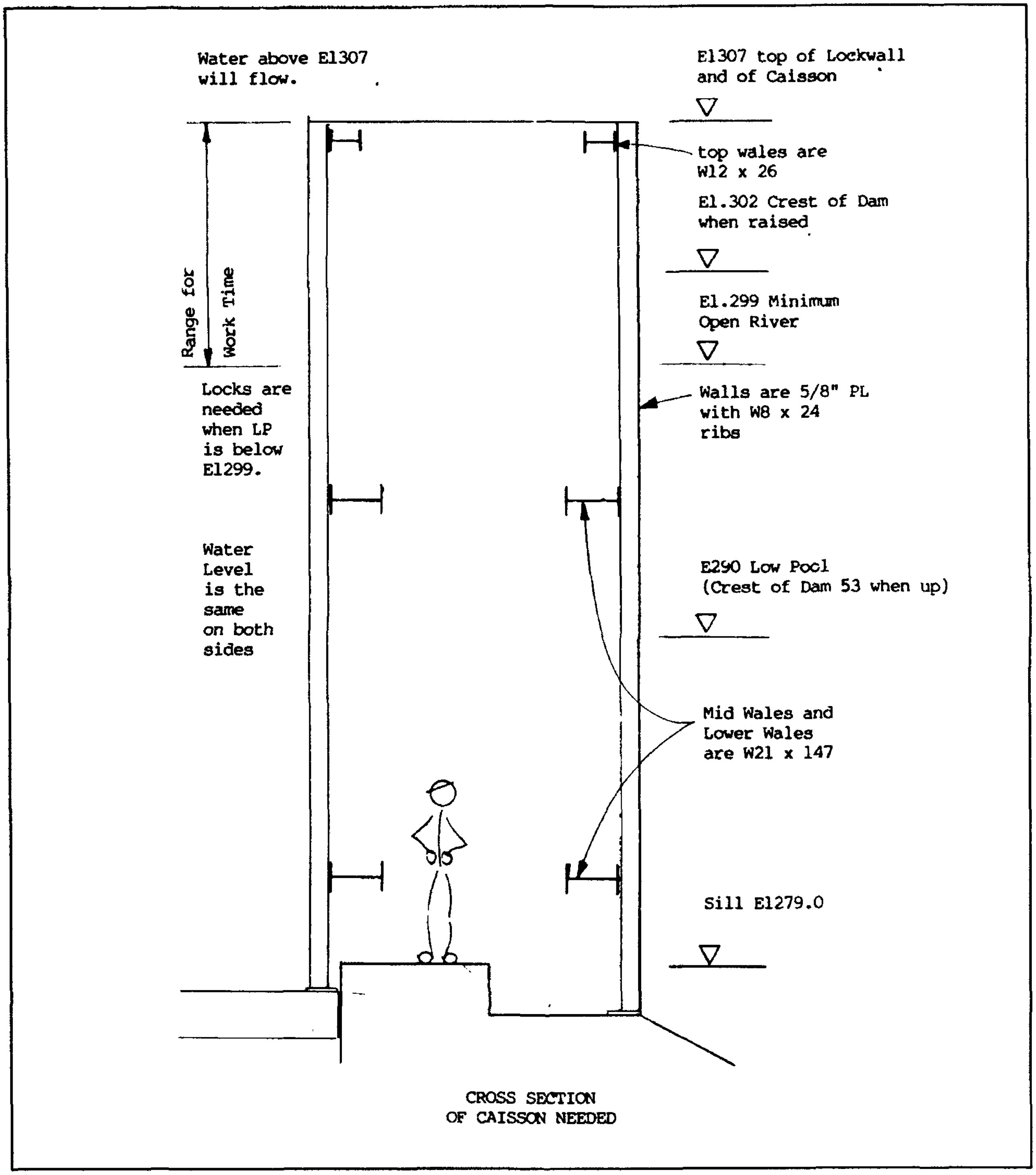

Figure 5. Cross section of caisson needed

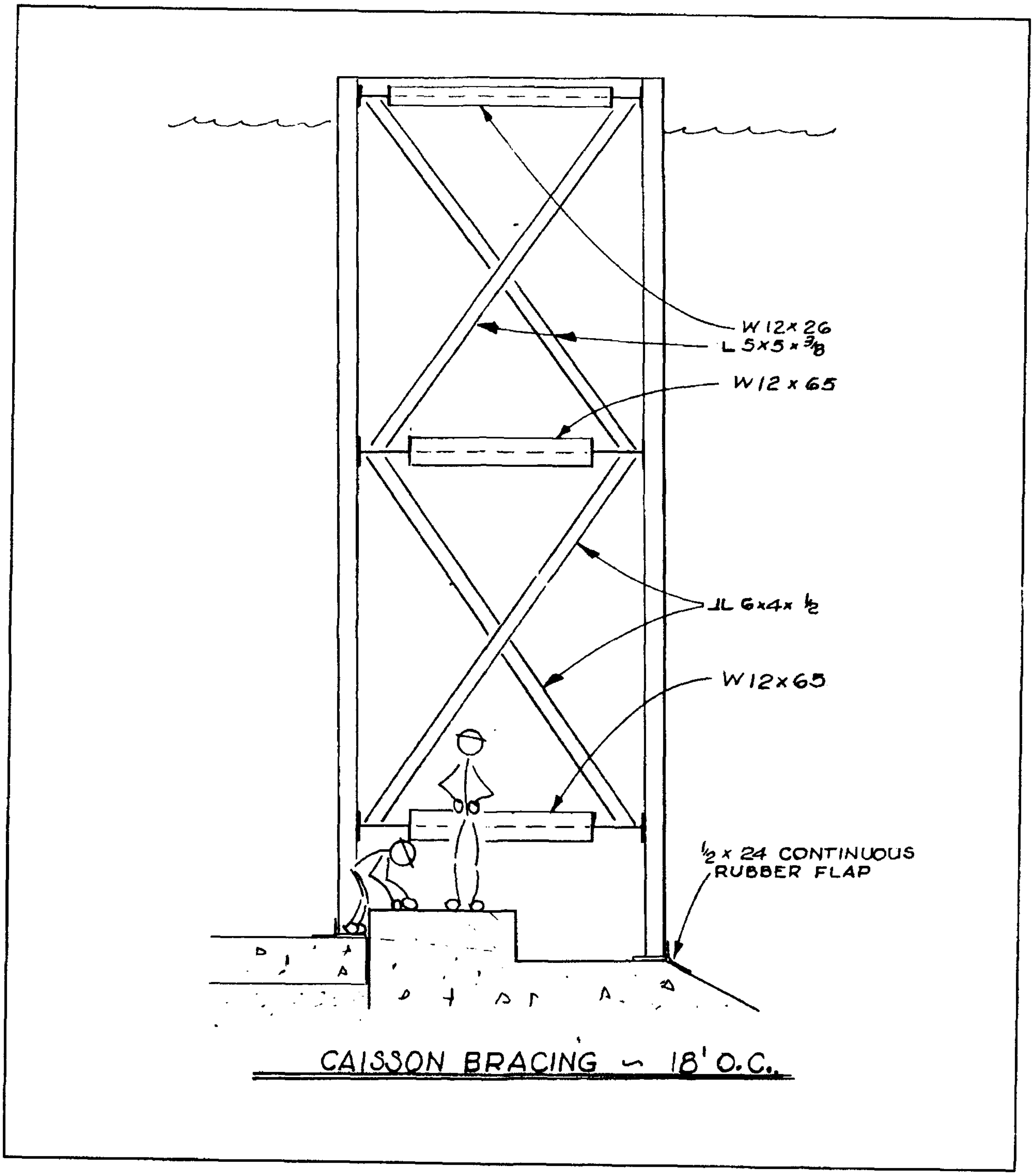

Figure 6. Caisson bracing ~ 18-foot O.C.

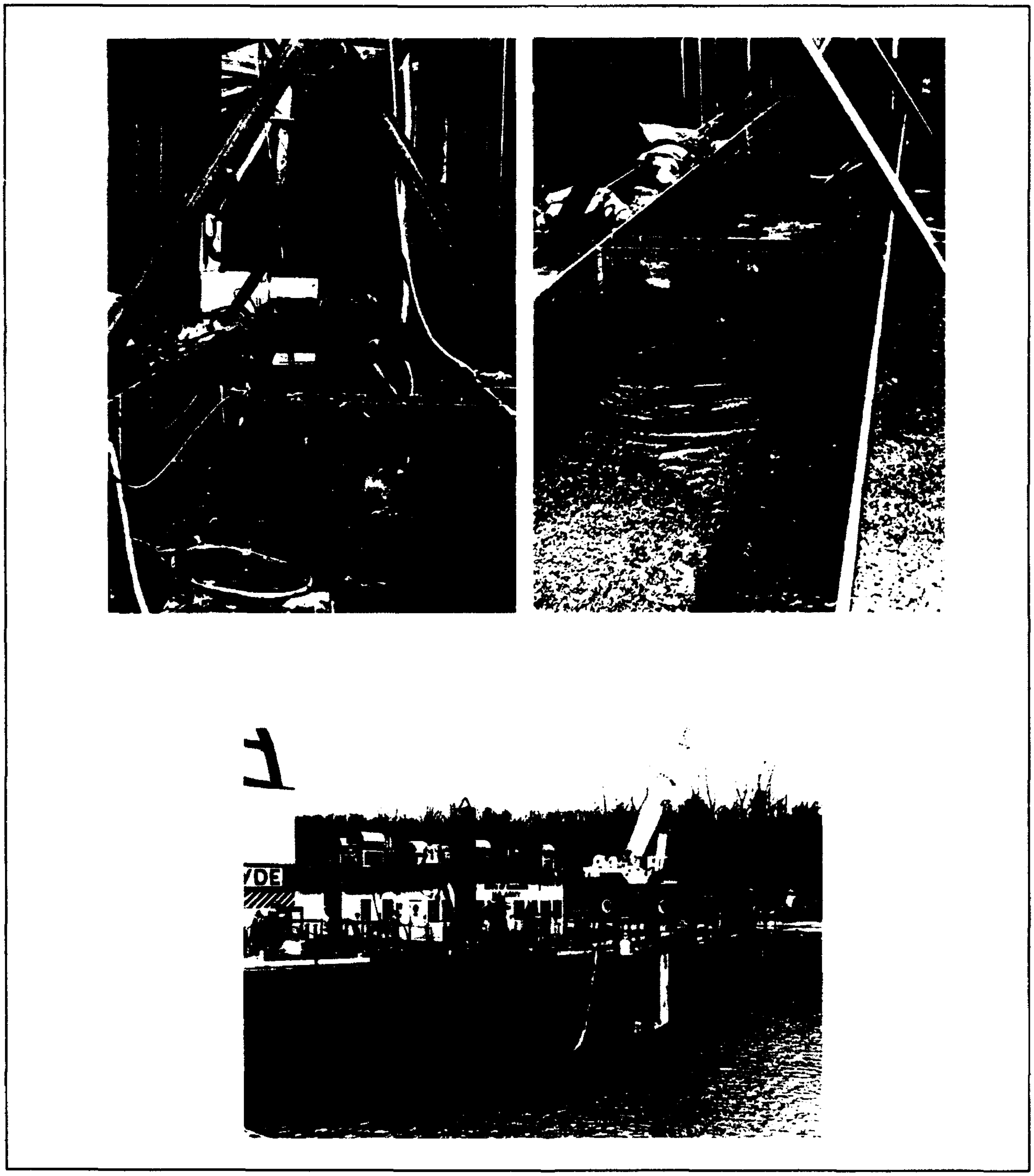

Figure 7. Photos of caisson

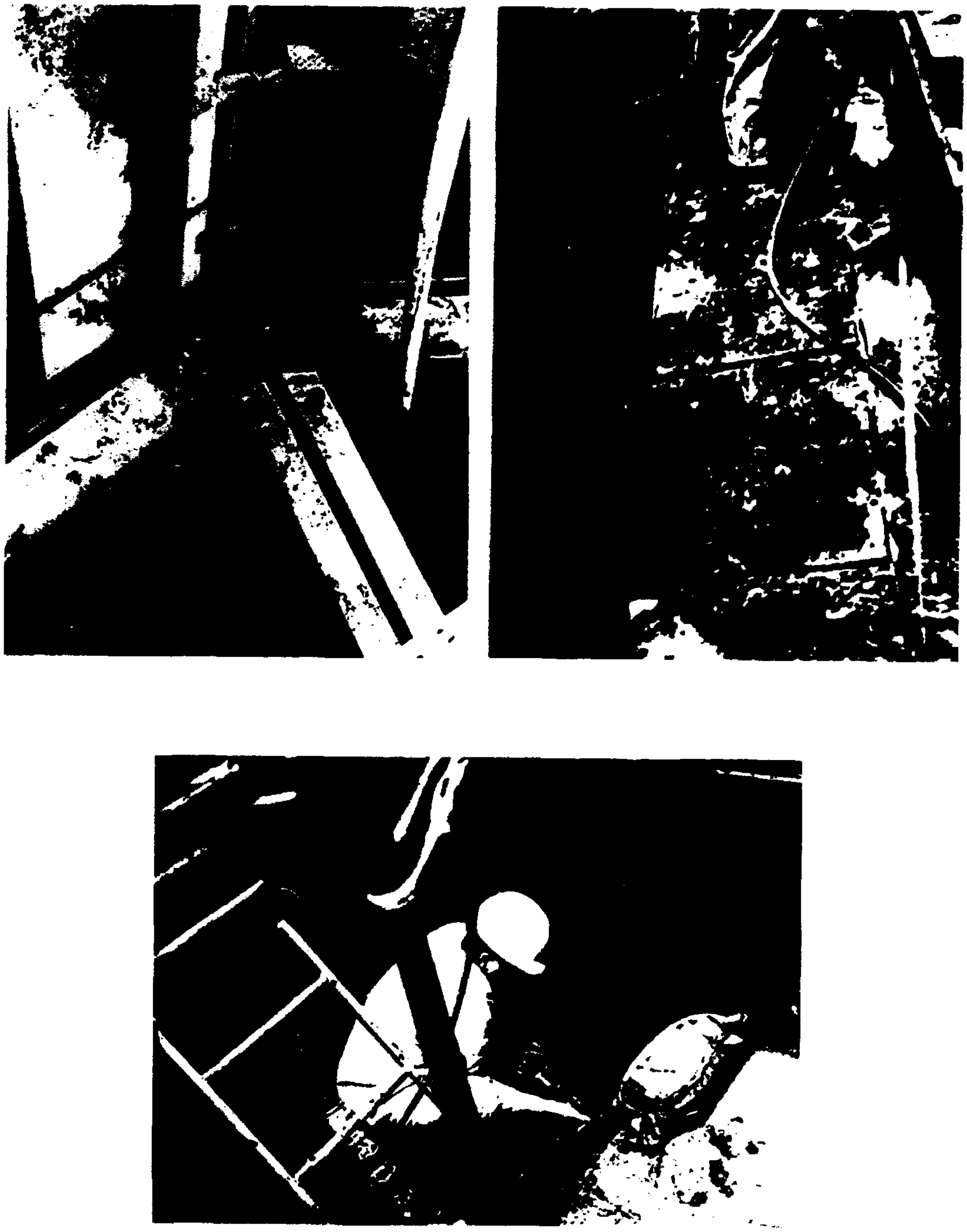

Figure 8. Photos of caisson

Rehabilitation of the Santa Ana Bridge Sandoval County, New Mexico

by
Lucy Ortiz[1]

Abstract

This paper addresses the rehabilitation of the Santa Ana Bridge, a 36-year-old timber structure, located in north central New Mexico. It is 609 ft long and is composed of twenty-nine 21-ft spans, mounted on three-pile timber bents. Over the years the bridge had been gradually downrated from the original H-15 live load rating to a live load rating of H-6. During a bridge inspection in July of 1989, it was determined that the bridge was in a serious state of distress and had visible signs of dry rot. An investigation consisting of making small borings into the wood members and measuring the extent of void present in the wood members was undertaken. The investigation concluded that 30 pile caps, 17 stringers, and 1 pile had sufficient dry rot to be classified as reject. The rehabilitation consisted of replacing the reject timbers, all of the cross-bracing on the bents, and patching the wearing surface.

Introduction

The Santa Ana Bridge is located in Sandoval County, approximately 7 miles west of Bernalillo, NM, at the Native American pueblo of Santa Ana. The bridge crosses the Jemez River and the upstream end of the lake formed by the reservoir at the Jemez Canyon Dam. The single-lane timber bridge provides access to the village of Santa Ana and the Santa Ana Indian Reservation to the north of the river. The bridge is approximately 609 ft long and is comprised of twenty nine 21-ft spans (Figure 1), supported on three-pile timber bents (Figure 2). The lane width is 11 ft. The original bridge, completed in 1953, is constructed of treated timber. Traffic on the bridge is restricted by a locked gate, and daily use is intended for the citizens of the Santa Ana Pueblo. Ceremonial events open to the public usually are held four times a year, and at these times it is not unusual for the traffic to be in excess of 750 vehicles in 1 day. During these events vehicular traffic is often backed up over the entire length of the bridge.

History of the Project

High river flows in 1957 resulted in failures of four individual pile bents. All four of the bents were replaced with new timber pile bents, and one bent was reinforced with a 50-ft timber pile on each side. Emergency repairs were again undertaken in 1958-59 when three bents were reinforced by adding 50-ft-long steel H-piles to the upstream and downstream piles of each bent. During a bridge inspection in 1979, many bridge elements were noted to

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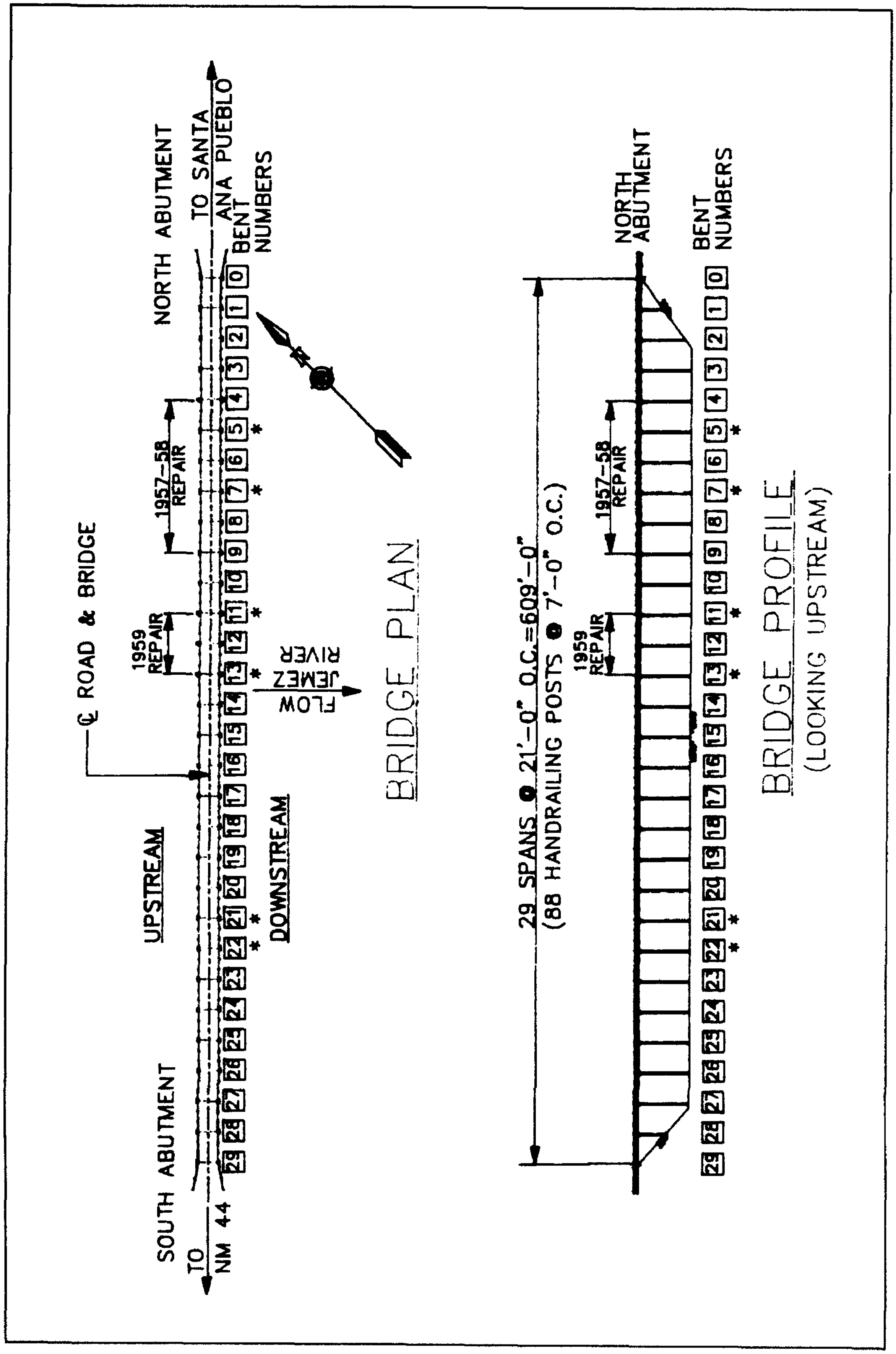


Figure 1. Diagram of Santa Ana Bridge

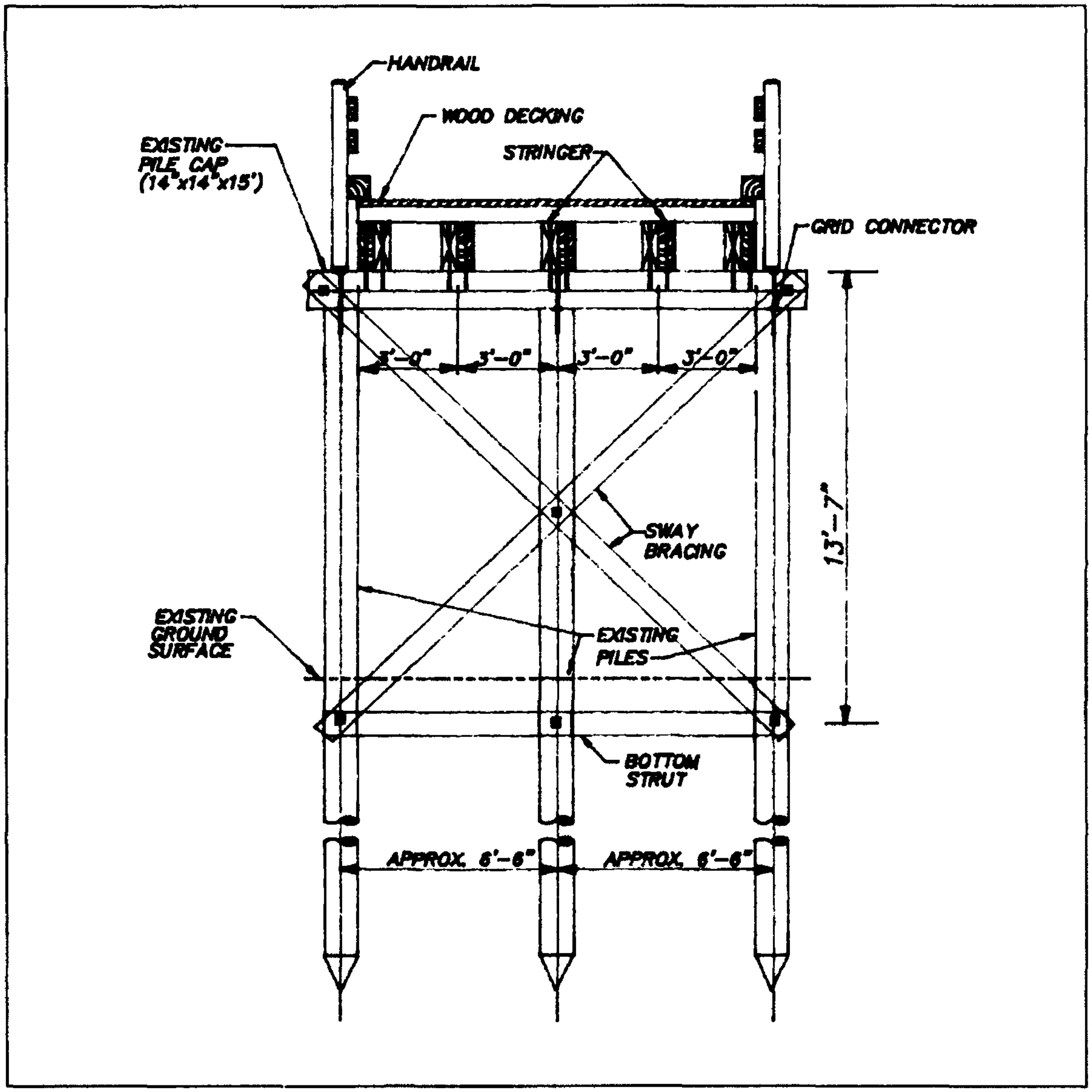


Figure 2. Three-pile timber bent—Santa Ana Bridge

have numerous horizontal cracks as well as significant deterioration of the wood members from the previous inspection. Based on this observation, a live load rating analysis was performed in accordance with the current AASHTO Standard Specifications for Highway Bridges. As a result of this analysis, the live load rating was reduced from the original H-15 to H-10 (Figure 3). The live load rating was further reduced to H-6 after another live load rating analysis was performed in 1981 when the number of cracks was noted to have increased. In Dec 1988, during a routine periodic inspection as prescribed by ER 1110-2-100 and "AASHTO Manual for Maintenance Inspection of Bridges" (AASHTO 1983), the pile cap at bent No. 21 (Figure 1) was noted to have a severe punching shear failure at the center pile (Figure 4). The cap was repaired by thru-bolting two steel channels on each side of the cap (Figure 5) and shimming the stringers to take the load off the timber cap. Bridge inspections in July and Aug 1989 revealed that the bridge was deteriorating rapidly. The bridge was subsequently closed and emergency repairs similar to those made to bent No. 21 were performed on bents No. 5, 7, 11, 13, and 22. However, these latest repairs were not

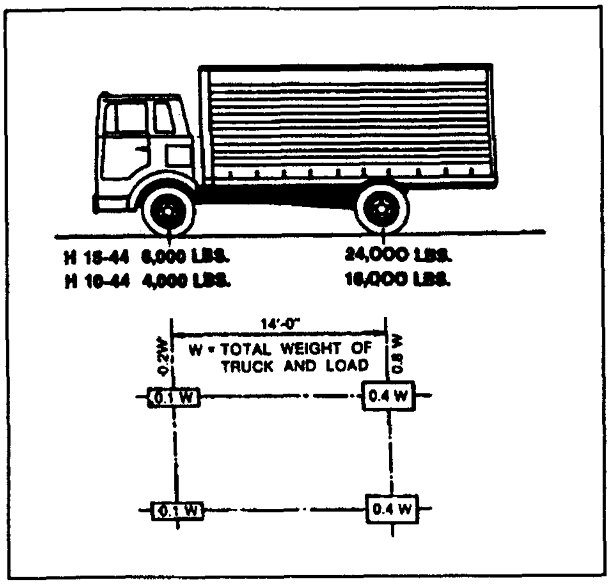


Figure 3. Standard H truck loading

Figure 4. Pile cap punching shear

sufficient in ensuring the overall safety and reliability of the bridge. It was determined that a comprehensive structural evaluation of the bridge was needed to determine the true condition of the bridge.

Figure 5. Steel channel repair

Evaluation of the Project

An engineering firm was contracted to perform an engineering study of the bridge. The study focused on four alternatives:

- Rehabilitate the existing bridge (H-15-44).
- Replace the bridge with a one-lane HS15-44 bridge.
- Replace the bridge with a one-lane HS20-44 bridge.
- Replace the bridge with a two-lane HS15-44 bridge.

Structural evaluation of the bridge

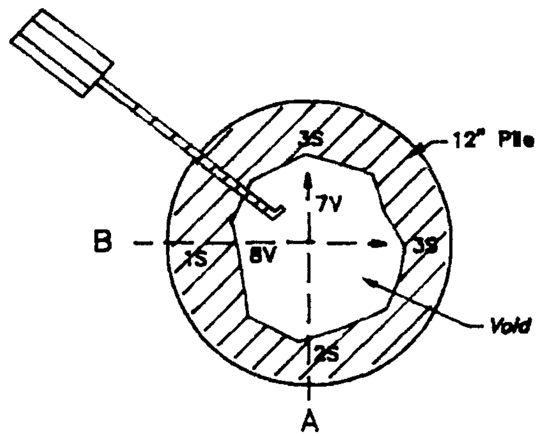
As part of the investigation of the first alternative, Osmose Wood Preserving, Inc., Madison, WI, a specialist in the field of timber rehabilitation, was subcontracted to perform a comprehensive inspection and evaluation of the structural integrity of each of the individual

bridge members. The inspection was accomplished by making 3/8-in. bores into the wood members. A calibrated probe was inserted into the hole and the size of the voids and the shell thickness of the wood member were measured. Voids caused by decay in the wood provided a measurable indication of the deterioration in the members. A timber member was considered "reject" if the amount of solid wood (cumulative shell thickness) was less than 6 in. as determined by the method shown in Figure 6. Based on this criteria, Osmose determined the following:

- Of 3u pilecaps, 22 had measurable levels of decay and 18 of these were considered "reject."
- Of 90 piles, 14 had measurable levels of decay and 1 of these was considered "reject."
- Of 145 stringers, 12 had decay and 4 of these were considered "reject."

In addition to this, Osmose also considered some features of the original construction to be the source of many of the existing problems. Osmose made several comments based on these features. Their comments are as follows:

- The transverse X-bracing is more ornamental than functional, since they do not contribute adequate stability to the joints between piles 1 and 3 (Figure 2) to the



Hole "A" Is Recorded As Follows: ....2" *Front Shell*
7" *Internal Void*
3" *Back Shell*

Hole "B" Is Recorded As Follows: ....1" *Front Shell*
8" *Internal Void*
3" *Back Shell*

To Determine A Reject In Piles, Use The Maximum Shell From Either A or B And The Minimum From Either A or B To Arrive At The Pile's Weakest Condition.

Example: 1" *Front Shell*
3" *Back Shell*
4" *Sound Wood Reject, Less Than 6"*

Note: In This Example, Pile Would Be Reject Since Sound Wood Is Less Than 6". The Measurements Shown Are For Example Purposes Only.

Figure 6. Sound wood determination method

cap. To accomplish proper bracing angles, the cap should be longer. Grid connectors are properly applied between the bracing and piles.

- The existing caps are too short (15 ft long) and too shallow (14 in. deep) for the intended purpose. Grid connectors should not have been placed between the pile and cap, instead a full contact joint should have been made and fastened together with drift pins. The grid connectors are promoting mechanical self-destruction of the caps.

- The piling, stringers, deck boards, guard rails, and asphalt deck surface were properly selected and installed and, with proper preventive maintenance, will continue to perform satisfactorily.

- In general, the hardware is in good, serviceable condition. Packing bolts of inadequate length were installed in the outside stringer ply joints - but, the countersinking has not yet caused major decay to gain a foothold at these points.

In conclusion, Osmose determined the structure could be economically repaired with minimal interruption to local traffic. As part of the rehabilitation, they recommended the replacement of "reject" or inadequate members to return the bridge to its original intended structural integrity. The members to be replaced were as follows:

- One pile, same dimensions.

- Thirty pile caps, 2-piece, bolted 7-in by 15-1/2-in. by 20-ft.

- Seventeen stringers, same dimensions. (Note: One additional stringer was added during construction.)

- All sway (cross) bracing, 4-in. by 8-in. by 24- to 26-ft.

- All A307 brace bolts of 3/4-in. diam.

After reviewing the structural evaluation of the bridge, it was decided to reduce the live load rating to H2.5. This was done to ensure that no vehicles heavier than a car or pickup truck were driven on the bridge. The bridge was additionally restricted to one vehicle at a time.

Evaluation of alternatives

Alternative #1—This alternative consisted of rehabilitating the existing timber bridge by replacing structural members which had been classified as **"reject"** by the inspection by Osmose Wood Preserving, Inc. Also, all timber members were to be treated with an environmentally safe chemical preservative, and the asphalt wearing surface was to be patched and resealed as needed. The preliminary construction cost estimate for this alternative was $250,000.

Alternative #2—This alternative involved replacing the existing timber bridge with a new one-lane HS15-44 bridge. The bridge was to consist of a 7-1/4-in.-thick cast-in-place concrete slab supported by prestressed concrete or steel beams bearing on pile bents constructed of steel H-piles and cast-in-place concrete pile caps. Assuming prestressed concrete beams, the preliminary construction cost estimate of this alternative was $950,000.

Alternative #3—This alternative involved replacing the existing timber bridge with a new one-lane HS20-44 bridge. The bridge construction was to be the same as that for alternative #2. The additional load capacity was determined to have a negligible impact on the cost and was therefore estimated at approximately $950,000, the same as alternative #2.

Alternative #4—This alternative involved replacing the existing timber bridge with a new two-lane bridge. The bridge construction was to be the same as that of alternative #2. Due to the additional lane, the preliminary construction cost estimate was $1,140,000, slightly higher than alternatives #2 and #3.

Recommendation

Several criteria were considered in the selection of the recommended alternative. The criteria were: 1) Construction Cost, 2) Operation and Maintenance (O&M) Cost, 3) Service Life Expectancy, 4) Traffic Considerations, 5) Constructability, and 6) Memorandum of Understanding with the Santa Ana Pueblo.

Construction cost—Of the alternatives investigated, the rehabilitation of the existing bridge, at a cost of approximately $250,000, was far less expensive than the other alternatives, which had a construction cost that began at $950,000.

Operation and maintenance cost—All the alternatives investigated had associated O&M costs. The alternative with the lowest O&M cost was construction of any of the new bridge alternatives with a cost of $800 per year. Rehabilitation of the existing bridge had an estimated O&M cost of approximately $2,360 per year.

Service life—The service life expectancy of all the alternatives was determined to be at least 50 years after the completion of construction with annual maintenance. This indicated that none of the alternatives would be expected to require extensive rehabilitation or replacement for 50 years. Although the existing bridge is more than 35 years old, Osmose Wood Preserving, Inc. stated it should provide service for another 50 years after rehabilitation.

Traffic considerations—The level of service for traffic during construction varied only slightly for the different alternatives. Rehabilitation of the existing bridge would only require the bridge to be closed for short periods of time and the bridge would be operable during nights and weekends. Construction of a new bridge would require the use of a low water crossing, during construction, if the same alignment was maintained or would require the use of the existing bridge if a new alignment was selected.

Constructability—None of the alternatives presented unusual constructability problems. The construction period was estimated to be 4 to 5 months for the new bridge alternatives and 2 months or less for the rehabilitation alternative.

Memorandum of understanding—The MOU with the Santa Ana Pueblo has been in effect since the bridge was originally constructed. The document agrees to provide the pueblo of Santa Ana with a safe, functional bridge. Construction of a new bridge represented an upgrade of the existing bridge and although the pueblo would have preferred this, it did not form part of the agreement.

After consideration of all of the above criteria, the decision was made to rehabilitate the existing timber bridge. This alternative fulfilled the requirement for safe access to and from the pueblo in the most cost effective manner and in the shortest period of time.

Bridge Repairs

On the basis of the recommendations made by Osmose, the following were removed and replaced:

- All original 14-in. by 14-in. by 15-ft pile caps were replaced with 2-piece, 7-in. by 15-1/2-in. by 20-ft pile caps that were bolted together (Figure 7).
- Eighteen stringers were replaced with new stringers of the same dimensions.
- The decayed portion of the bad pile was spliced with a new section of timber pile of the same diameter.
- All cross-bracing members, except for the bottom strut, were replaced with new longer cross-bracing of the same cross-sectional dimensions (Figure 7). This bracing connected the pile caps to the piles.

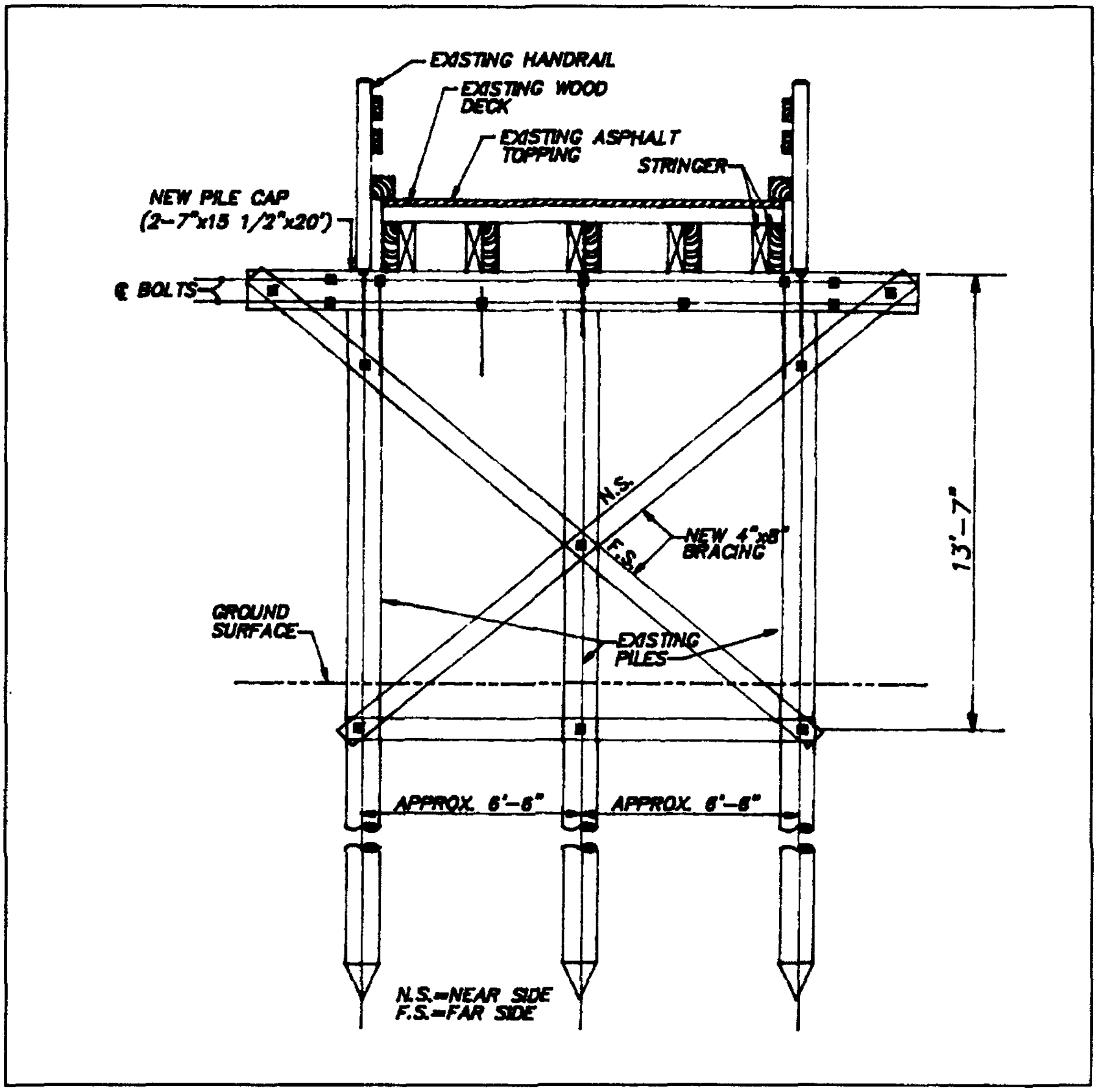


Figure 7. Rehabilitated pile bent

Construction

Notice to proceed for the rehabilitation of the existing bridge was given on 1 March 1990 and construction was completed on 25 June 1990 at a total cost of $228,729.

Acknowledgements

I would like to thank the entire design team, from design project manager to construction project manager, for the support they provided during the development of this report. My appreciation goes also to those colleagues who kindly reviewed drafts of my efforts. Last, to the rest of the Architectural/ Structural Engineering Team: "This is for you!"

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Nonlinear Incremental Structural Analysis and Fracture Mechanics—A Logical Link

by

Barry D. Fehl, PE,[1] *and Dr. Kevin Z. Truman*[2]

Abstract

A discussion of the need for a fracture mechanics and nonlinear material based analysis tool for use in the design of massive concrete structures is presented. A brief history of the Corps of Engineers' development in the fields of fracture mechanics and nonlinear incremental analyses is used to explain the benefits to be derived from linking these two distinct types of analyses. The nonlinear incremental structural analysis (NISA) is required in many instances through ETL 1110-2-324 while a separate fracture analysis is required for certain structures through ETL 1110-8-16(FR). The linking of the two procedures will provide a means of evaluating newly constructed massive concrete structures with regards to thermal effects which may cause early age cracking. These cracks can then be checked for their stability and effects on the postcracked structure. The new algorithm is presented and discussed with the use of a flowchart and a simple example. Each step of the algorithm is discussed with respect to its purpose, compatibility, and practical implementation.

Introduction

Problems in dealing with the temperature loadings that act on massive concrete structures and the associated cracking resulting from these loads have been addressed within the Corps of Engineers for several decades. Research has now advanced to the point where predictions of temperature loads can be made as well as the capability of locating a discrete crack which may occur within a concrete structure. Therefore, it seems logical to create a link between the discrete crack model and temperature load analysis techniques in order to predict cracks due to temperature loads imposed on the structure.

The theory for implementing the link between prediction of temperature loads and a discrete crack model will be presented. Temperature loads will be predicted using a nonlinear incremental structural analysis (NISA) by methods currently being used within the Corps of Engineers. A fracture mechanics based cracking model which is capable of predicting both the length and orientation of a crack will be used to predict cracks occurring in the concrete. A simple example was used to assist in the development of the procedures and to implement the proposed theory. Development of the proposed link between a NISA and a fracture-based cracking model is an important part of helping the Corps of Engineers achieve the total design quality it desires.

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Background

The NISA currently being used within the Corps of Engineers first began its development in 1983 through a study performed by the US Army Engineer Waterways Experiment Station (WES) and funded by the St. Louis District (Bombich, Norman, and Jones 1987). The new method replaced a method used since the late 1960's for predicting stresses and strains due to temperature loadings. The major consequences of this particular study were that the finite element code ABAQUS (Hibbitt, Karlsson, and Sorensen, Inc. 1986) was selected for performing the analyses, methods for performing a NISA were developed, and a constitutive model capable of implementing time dependent material properties was incorporated.

Further development of the NISA was undertaken in 1987 through another study done by WES and Washington University (St. Louis, MO) and funded by the St. Louis District. This second study further developed methods used in the first study. Some of these developments were the limitation of element size for the heat transfer analysis and enhancement of material properties through testing. A better understanding of the analysis was also achieved through parametric studies of soil, creep, shrinkage, refinement of the finite element mesh at edges of the model, and gap elements (Truman, Petruska, and Fehri, in preparation). Finally, analyses were also performed which indicated that lift heights in the auxiliary lock of Melvin Price Locks and Dam could be increased for a construction cost savings of $1.2 million (Truman, Petruska, Fehri, and Fehl 1991).

As a result of these studies, guidance for performing a NISA was developed in the form of ETL 1110-2-324, "Special Design Provisions for Massive Concrete Structures." ETL 1110-2-324 provides guidance for the procedures used in performing a NISA, input parameters to be used in a NISA, and how results from a NISA should be used and presented. Guidelines are also given for determining when a NISA should be performed and what types of structures should be analyzed using a NISA. Also contained in the ETL are examples from Melvin Price Locks and Dam and the Portuguese Arch Dam. The primary analytical tool for performing a NISA is the computer code ABAQUS.

Fracture mechanics is a theory which uses the energy at a crack tip and the resistance of the material to determine crack propagation. Much of the initial groundwork of fracture mechanics was laid in the late nineteenth and early twentieth centuries by mathematicians. Not until the 1950's when Irwin began implementing the works of these mathematicians did it become known as fracture mechanics. Early research in the area of fracture mechanics was primarily on metals, but in the 1970's researchers began implementing the methods found to work so well on metals and applied them to concrete.

Dr. Victor E. Saouma along with Dr. A. R. Ingraffea reintroduced a computer fracture model which was originally investigated in the 1960's but was unsuccessful due to the lack of adequate computing capability. Using the computer model he helped develop, Dr. Saouma has performed a considerable amount of work for the Electric Power Research Institute (EPRI) in the area of concrete fracture. His work for EPRI includes a report on the fracture mechanics of concrete dams. The report is made up of three volumes: (a) the first introduces the concepts of fracture mechanics and fracture mechanics of concrete dams, (b) the second describes experimental and numerical studies being undertaken at the University of Colorado in Boulder in the area of fracture of concrete dams, and (c) the third is a user's manual for the computer program CDAM, developed by Dr. Saouma for performing fracture analyses (Saouma, Broz, Bruhwiler, and Ayari, in preparation).

Criteria for fracture of concrete have been developed for Corps of Engineers structures from Dr. Saouma's research. The criteria are contained in ETL 1110-8-16(FR), "Fracture Analysis of Concrete Hydraulic Structures," which presents a brief introduction into fracture mechanics and fracture of concrete.

There are also examples, including a fracture analysis of a lock monolith at Locks No. 27 on the Mississippi River. The program CDAM is the primary tool used in performing a fracture analysis.

Purpose

It would be reasonable to ask why a link between NISA and a fracture mechanics crack model is needed. The primary reason is that guidance is currently in place which requires NISA and fracture mechanics analyses to be performed. ETL 1110-2-324 guidance on NISA requires the analysis to be performed if economics indicate that money can be saved, if a conventional structure has shown past poor performance, or when a structure without precedent is designed. ETL 1110-8-16(FR) on fracture mechanics analysis requires an analysis when conventional methods show that an existing structure is unstable or that cracking is predicted. Both methods have been determined to be viable tools, and the obstacle which prevents a fracture mechanics analysis on a newly designed structure is a means of incorporating the model with the design tool used to evaluate a newly constructed massive concrete structure.

Other reasons for linking a NISA to a fracture-based cracking model exist as well. An example contained in ETL 1110-8-16(FR) featured the use of a fracture mechanics analysis to evaluate a monolith at Lock No. 27 on the Mississippi River which had been shown to be unstable for the dewatered condition using conventional methods of analysis. The fracture mechanics approach indicated that the monolith was not unstable. While not totally conclusive, the lock was dewatered in February 1991, and the monolith performed satisfactorily, indicating that the fracture analysis was representative of what was actually occurring in the field. Since the fracture mechanics approach appears to be successful in this case, it is reasonable to assume that it could perform equally as well in conjunction with a NISA.

Another reason for using a fracture-based crack model is that in the past a discrete crack model was the preferred crack model but the computing power was not available to accommodate this type of model. This lack of computing power necessitated the introduction of the smeared crack model which is currently used when performing a NISA. In recent years, though, disk space, speed of calculations, and memory have increased dramatically to a point where a discrete crack model, such as the fracture model, is a very viable option.

Since computers have now reached the level needed to perform an analysis using a discrete crack model, it makes perfect sense to implement such a model. While a smeared crack model is a valuable tool, a fracture-based crack model is more realistic since it models the actual crack instead of reducing the stiffness of the finite element model as done by a smeared crack model. The fracture-based model also provides more useful and practical results such as crack length, crack orientation, and crack opening displacement. The mesh refinement for the initial mesh is much less critical for a fracture-based cracking model as compared with a smeared crack model, particularly when considering the thinner members of the structure. If a thin member in a structure is not modeled with the proper number of elements in a smeared crack analysis, it could create an instability problem in the member, and ultimately in the structure, which may or may not exist due to the loss of stiffness of an element. A discrete crack may not have the same effect because the crack can begin at a length which is smaller than the size of the element. In addition, much research and testing of the fracture-based model for concrete have been performed such that a level of reliability exists which allows it to be used with confidence when evaluating concrete structures.

A final reason for developing a link between a NISA and a fracture mechanics crack model is to provide opportunities in the future for the Corps of Engineers to move ahead in the design of massive concrete structures. Current criteria for cracking are very restrictive. The linking of a NISA and a fracture-based model would allow the Corps of Engineers to

consider criteria such as limiting crack length or the crack opening displacement, leading to more economical structures without a loss of reliability.

Methodology

A detailed explanation of the methodology for development of a NISA link to a fracture mechanics based crack model is presented below. The conceptual theory will be presented to provide insight into how the method will function once the link is complete. A discussion will then follow identifying the obstacles which are known or expected to occur in the developmental process. Probable solutions to the obstacles will also be provided, and known successful portions of the methodology will be discussed. Included in these discussions will be reference to a problem which has been used in developing the components of the methodology.

The methodology for accomplishing a NISA with a fracture-based cracking model can be described in a general form using the flowchart shown in Figure 1. The first three steps of the flowchart are to build a finite element model, perform a heat transfer analysis, and perform an incremental nonlinear stress analysis, including the use of the smeared crack model. These three steps basically constitute a NISA and should be performed within the guidelines set forth in ETL 1110-2-324. Once the NISA has been completed, a review of the results for indications of cracking predicted by the smeared crack model should be made for determining the time and location at which the crack occurred as has been noted in step four. Once the time step where cracking will begin has been determined, then the previous time step of the NISA can be used as a starting point for the fracture analysis. As indicated in step five of the flowchart, once the time step for beginning the fracture analysis has been determined, the displacements must be obtained for input into the program which evaluates the crack. Prior to using the crack evaluation program, a remeshing of the finite element mesh must be made at the location previously determined in step four, and displacements must be assigned to the new nodes resulting from the remeshing as indicated in steps six and seven.

Step eight in Figure 1 is the actual analysis of the model using the fracture analysis program developed by Dr. Victor Saouma. Once the fracture analysis is complete, a determination can be made from the results of the analysis as to whether the crack has arrested or will continue to propagate. If at step nine it is determined that the crack has arrested itself, then the analysis is complete. Similarly, if it is determined that the crack will continue to propagate, then the analysis moves to step ten, which requires the finite element mesh to once again be revised and remeshed based on the fact that the crack will propagate. In step eleven, the temperatures for the next time step of the NISA will be retrieved, and temperatures will be assigned to the new nodes created in the remeshing. These temperature values are then used in step twelve to perform the next time step of the stress analysis. Once the stress analysis is complete, the procedure returns to step seven for extraction of the nodal displacements for use by the fracture analysis program. Steps seven through twelve are then repeated until the crack stops propagating or until the structure fails.

As might be expected, many obstacles must be overcome before the process described above is functional and can be used by the engineer with a relative amount of ease. A simple example problem was developed to assist in development of the method in identifying points in the procedure which will create difficulties. The finite element mesh of the problem is shown in Figure 2. The support conditions shown are the conditions existing 2 days after the second lift of concrete was placed. This structure was developed in order to create a crack at the center line of the bottom of the second lift. It was also anticipated that the crack will arrest itself prior to reaching the top of the lift due to arching action. This type of arching action typically occurs above culvert openings in locks. In addition, its dimensions were felt to be of significant width and height to represent the effects of a massive concrete structure.

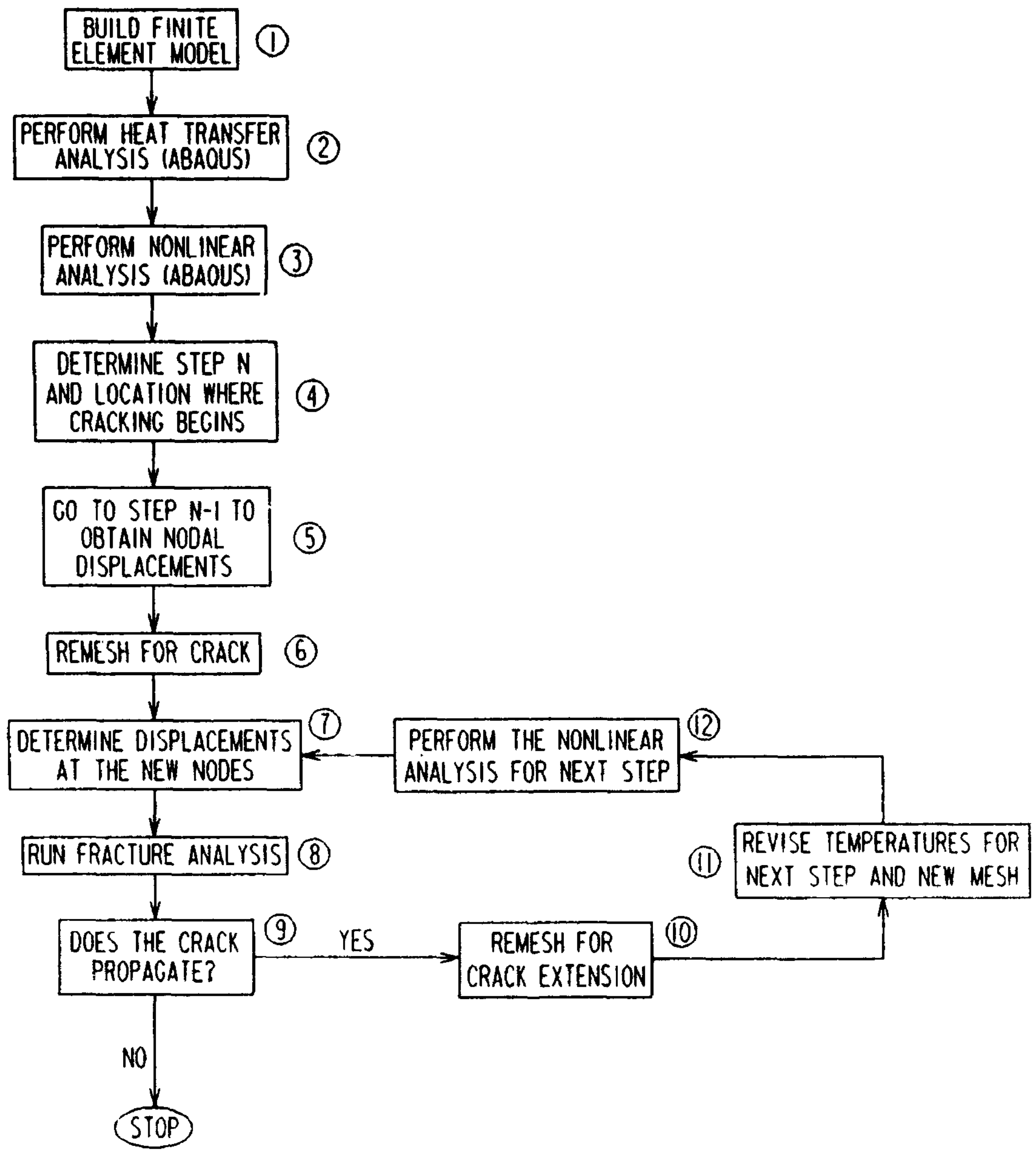

Figure 1. Flowchart for performing an analysis using NISA and a fracture mechanics based crack model

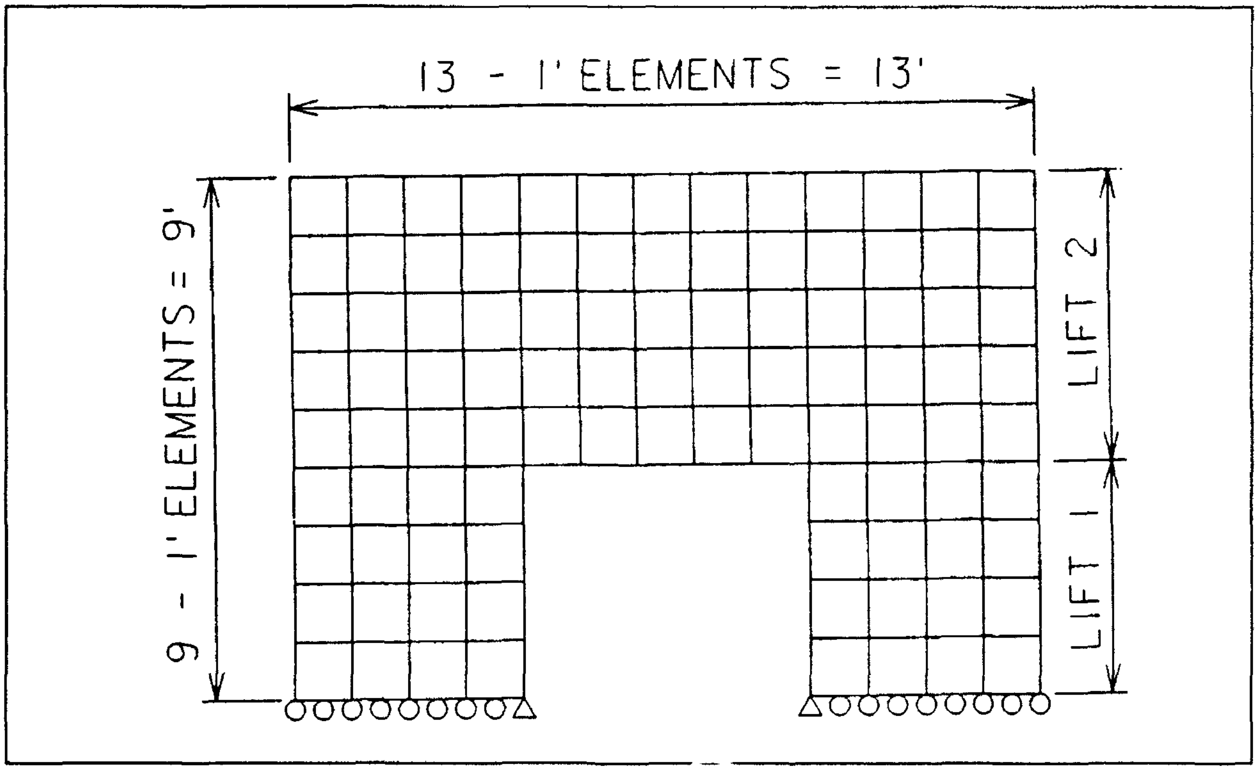

Figure 2. Finite element mesh of example problem

One of the difficulties that exist with the current capabilities is the remeshing that must be done around the crack. Discretization to accommodate the crack in the problem being used was very tedious and time consuming. The remeshing required 23 additional elements as seen in Figure 3. An automatic remeshing subroutine will be obtained or developed sometime in the near future in order to avoid further delays due to the remeshing process.

Another problem which has been apparent from the outset is that when the finite element model is remeshed to account for the discrete crack, no temperature values exist for the new nodes introduced as a result of the new mesh. This is noted in step eleven of Figure 1. Currently the newly defined mesh must be reanalyzed from the initial time step to obtain temperatures at these nodes. The anticipated solution to this problem is to develop a subroutine which will assign temperatures to the new nodes based on the temperatures of the existing nodes surrounding the new nodes by using an interpolation algorithm.

A similar problem is the need to assign displacement values to nodes as indicated in steps seven and twelve of Figure 1. Once again, the only means of obtaining displacements at nodes due to a new mesh is a re-analysis for the new mesh from the very first step. The solution is also similar to the problem discussed earlier in that a subroutine will need to be developed which will interpolate displacement values for the new nodes based on displacements at existing nodes. Further manipulation may be required at step twelve in order that the ABAQUS analysis may be continued from this point in the analysis.

The most significant of the hurdles to overcome is the link between the fracture model and ABAQUS, of which the aforementioned subroutines will be components. To perform a complete analysis which would include a NISA and fracture analysis would require extensive manipulation to go from one to the other. Current plans are to develop a command control file which will perform the manipulations required to go from the NISA to the

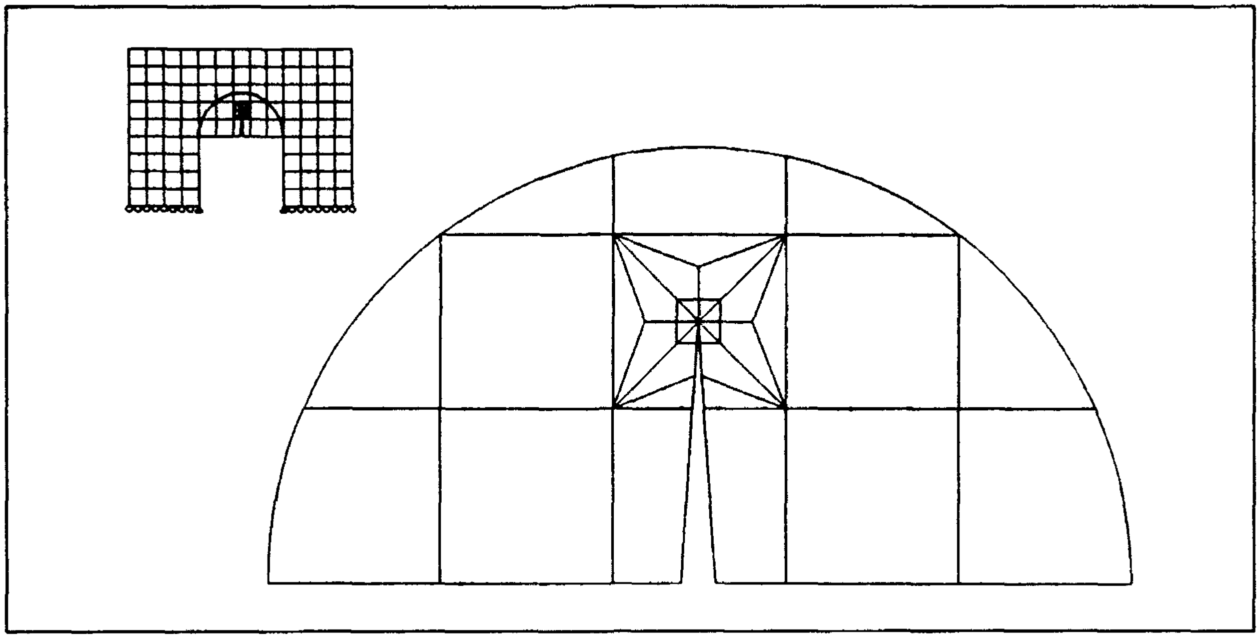

Figure 3. Remeshing required around crack

fracture analysis and likewise back to the NISA so that the analyst will only be required to initiate the analysis. This command control file will contain all the necessary translators, commands, and interpolation subroutines necessary to complete a NISA with a fracture-based cracking model. Another alternative that will be evaluated is the possibility of arranging for the cracking model to become a part of the ABAQUS computer code.

While many difficult tasks remain before the NISA and fracture model link will be complete, successful progress has been made in the development of this link. One significant piece of progress is the development of a stand-alone program for performing the fracture analysis. Portions of the CDAM program developed by Dr. Saouma were taken and developed into a separate program which has been successfully run. In addition, ABAQUS results were successfully transferred and used by the crack model.

Another accomplishment achieved was a successful ABAQUS analysis on a model which had been remeshed for a crack. While this would not appear to present a problem, some problems with excessive movement of the concrete into the crack occurred, creating a problem that could not reach a convergent solution. In order to obtain a convergent solution, the dead weight of the concrete was not activated until the beginning of the third day after the lift was placed. Further investigation into optimizing the analytical procedure with a crack present will be undertaken.

Still another positive aspect of the research completed is the fact that while many obstacles to completing the link between a NISA and a fracture analysis have been presented, there have been an equal number of solutions presented which can be fully expected to work. Problems without solutions are a deterrent to completing an objective, but problems with solutions provide a positive challenge that gives initiative and motivation.

As previously mentioned, the finite element model shown in Figure 2 is being used in the development of the NISA and fracture analysis link. Using this finite element model with varying boundary conditions and input parameters has revealed several items that should be accomplished before proceeding very far into the research. First, as was mentioned before, is the need for a mesh generator to perform the task of generating the new nodes and elements around a crack. A great deal of time can be

saved and much frustration avoided by implementing such a tool. In a similar light, a mechanism is needed which can automatically reduce the output produced by ABAQUS in a time-history analysis to a useful set of numbers. Finally, in the ABAQUS analyses which contain a crack, interface elements may be needed within the crack to prevent the crack from closing past a point which is physically possible. An interface element could be placed in a crack allowing the crack to open but prevent it from closing beyond a point where the faces of the crack are in contact.

Conclusion

Progress dictates that the state of the art in analysis of massive concrete structures move towards a combined analysis tool utilizing NISA methods and a fracture-based cracking model. The components for achieving such an analysis are available and require only time, ingenuity, and development to make the components act as a unit. If the Corps of Engineers wishes to maintain its standing as a leader, research into areas such as combining NISA methods with a fracture-based cracking model must continue.

It is important to remember that established criteria within the Corps of Engineers already mandate the use of NISA and fracture analyses in given cases. It is equally important to remember that there are features in a discrete crack model which provide advantages over a smeared crack model to the engineer who may be utilizing this analytical tool. Finally, the proposed linking of the analysis procedures will provide a means for improving the economics and reliability of massive concrete structures for years to come.

The benefits of providing the combined procedure include the ability to predict the exact locations of cracks, determination of crack propagation distance, and determination of the crack opening displacement. In addition, a fracture-based cracking model will not overemphasize the importance of a crack as can be done with a smeared crack model when the finite element mesh is too coarse. It is anticipated that other benefits will become apparent as development of the procedure continues, but the benefits presented along with the Corps of Engineers' efforts to achieve total design quality provide valuable incentive for proceeding with research in the area of linking procedures of NISA to a fracture-based cracking model.

Acknowledgments

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Nonlinear, Incremental Structural Analysis of Olmsted Lock

Sharon Garner, PE,[1] *and Chris Merrill, PE*[2]

Abstract

The proposed Olmsted "W" frame lock configuration raised constructibility concerns within the US Army Engineer District, Louisville, due to its unique and massive cross section which varied from the traditional "U" frame lock shape. The Structures and Information Technology Laboratories at the Waterways Experiment Station were requested to jointly perform a nonlinear, incremental structural analysis (NISA) to determine the constructibility of the "W" frame lock section. Two proposed concrete placement schemes, "strip" and "block," were analyzed using thermal and mechanical properties of the two most likely concrete mixtures. The ABAQUS finite element program, coupled with a time-dependent material model, was used for two-dimensional plane stress and strain analyses and for a three-dimensional analysis. Monolithic behavior of both schemes was assumed for the analyses. Results indicated that thermal stresses from both mixtures were not sufficient to cause construction difficulties or excessive long-term cracking in either of the placement schemes. Both placement schemes showed high tensile stresses at vertical construction joints; however, these stresses were less critical for monolithic behavior in the "strip" placement scheme.

Introduction

Olmsted lock and dam will be located on the Ohio river near its confluence with the Mississippi River and is intended to replace locks and dams 52 and 53. The typical proposed lock section monolith is a pile-founded "W" frame section, approximately 326-ft wide and 76 ft long with twin 110-ft-wide lock chambers separated by a 52-ft-wide center wall. This unique and massive cross section varied from the traditional "U" frame shape and raised constructibility questions within the US Army Engineer District, Louisville. Two possible placement methods, the "strip" and the "block," were devised to maximize concrete placement and are presented in Figure 1. The Structures and Information Technology Laboratories at the Waterways Experiment Station were requested to jointly perform a nonlinear, incremental structural analysis (NISA) to determine their constructibility. The project was divided into three phases. Phase I consisted of a concrete material study, mixture proportioning investigation, and thermal and mechanical properties testing of mixtures selected as the most likely concrete mixtures to be used in construction. Phase II consisted of NISA's of two proposed placement schemes using material and mechanical properties of the two most likely concrete mixtures. Phase III has not been completed, but will consist of

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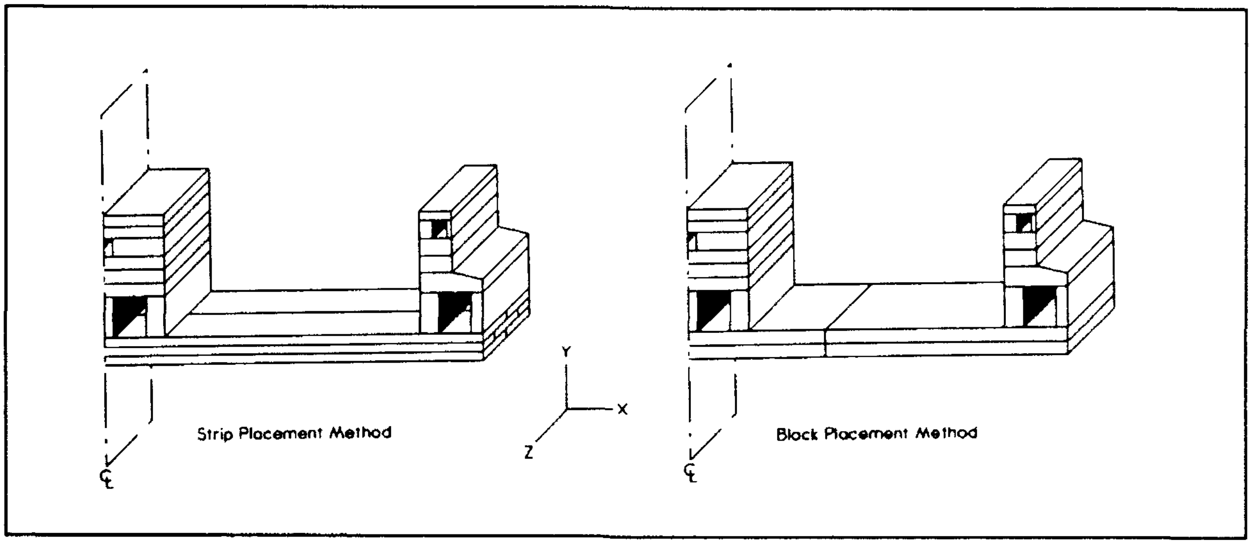


Figure 1. Proposed placement methods

an earthquake analysis of the structure. This presentation outlines work accomplished in Phase II.

Concrete Mixtures

A matrix of 12 mixtures was developed to cover the range of expected combinations of water-cement ratio and fly ash replacements for field use and to make use of both Class F and Class C fly ashes. Of these 12 mixtures, mixtures 6 and 11 were chosen for use in the analyses. Both mixtures used Type II, low alkali ASTM C 150 (ASTM 1990) portland cement with natural river sand including a small amount of filler sand for fine aggregate and coarse aggregates consisted of limestones. Mixture 11 had a water-cement ratio of 0.45 and used a Class C fly ash for a 0.50 fly ash replacement ratio. Mixture 6 had a water-cement ratio of 0.40 and used a Class F fly ash per ASTM C 618 (ASTM 1990) for a 0.40 fly ash replacement ratio. The design and selection of the concrete mixtures is described by Hammons et al. (in preparation).

Properties

Thermal properties for both concrete mixtures were based upon the results of tests conducted in Phase I. An initial heat transfer analysis of a column of soil indicated that a 20-ft depth of soil should be included in the analyses. Existing soil, described as a McNairy 1 clay, was to be excavated to a depth of 5 ft and backfilled with one of three possible materials: No. 57 limestone, compacted river sand, or quarry-run limestone. Thermal properties for the soil were estimated based on available soil properties. Preliminary heat transfer analyses indicated use of No. 57 limestone resulted in the largest temperature rise in the floor slab. This foundation condition was deemed the "worst case" and was used in subsequent heat transfer analyses. A time-dependent creep model was used in the stress analyses. Mechanical properties required by this model are: time dependent shrinkage curves, age-related modulus and creep functions, 3-day elastic modulus, Poisson's ratio, 3-day compressive strength, tensile strain capacity, and the coefficient of thermal expansion. These properties were obtained through material testing conducted in Phase I.

Two-Dimensional Models

Two sections were selected for the initial two-dimensional (2-D) analyses: 1) a typical W-frame section transverse to the axis of flow; and 2) a typical floor section parallel to the axis of flow. Lock monoliths were assumed symmetric relative to the axis of flow.

Eight-node quadrilateral and six-noded triangular elements were used. These elements provide nonlinear fields for temperatures and displacements. Standard integration was used for all heat transfer analyses, and reduced integration was used in the stress analyses. Element size was based upon two constraints. The first constraint results from the type of integration procedure used in the ABAQUS transient heat transfer algorithm, a relationship existing between the minimum usable time-step and the distance between nodes (ETL 1110-2-234). The second constraint on element size is based on lift dimensions. A minimum of two elements is required to define the stress distribution across a section. Due to these restrictions, floor elements were 24 in. high. With a few exceptions, wall element size was restricted to a maximum of 30 in. by 30 in. Grids used in the 2-D analyses are shown in Figure 2. Stress analyses were made using both plane strain and plane stress elements. Because of the condition imposed by the plane strain elements, restraint of out-of-plane strains is enforced, and incremental stresses due to the restraint of these strains are calculated. However, total restraint is not a realistic condition, and predicted out-of-plane tensile stresses can be excessive. The material model uses an interactive stress/strain cracking criteria with a maximum allowable stress of $2f_t'$ under a zero strain condition (Norman, Campbell, and Garner 1988). Cracking due to these out-of-plane stresses are not based on realistic conditions and can result in numerical nonconvergence as cracks continue to open.

Three-Dimensional Model

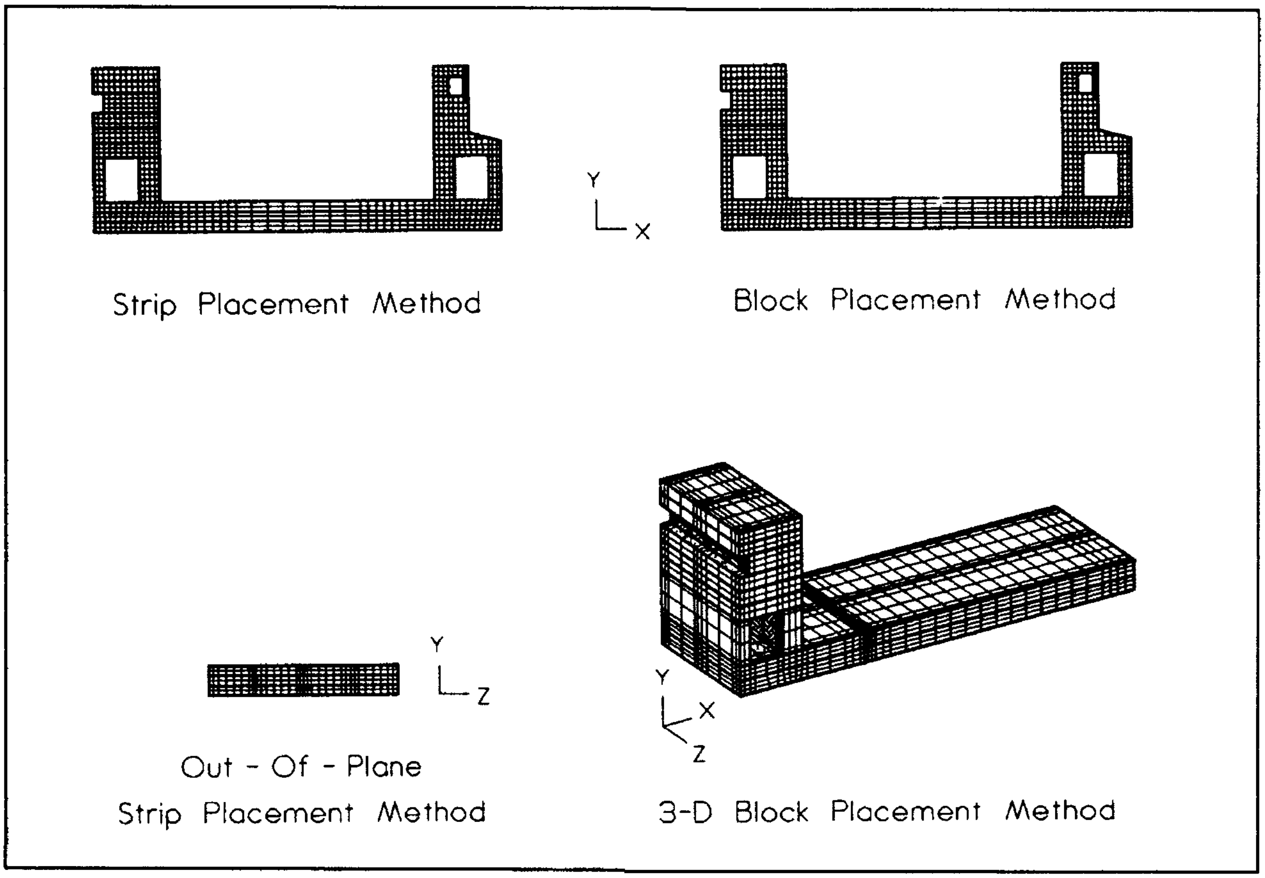
A quarter section was chosen for the three-dimensional (3-D) analysis. To simplify the analysis, this section included only the floor and the center wall. The maximum element dimension in the direction of heat flow was extended to 36 in., and element dimensions along the axis of flow were limited only by pile spacing except at joints and at the end face of the monolith. Twenty-node brick elements were used for the heat transfer and stress elements. Full integration was used in the heat transfer analysis and reduced integration was used in the stress analysis. The 3-D grid of the structure is shown in Figure 2.

Construction Conditions

The minimum lift placement interval for the analysis was 5 days. Longer placement intervals were used between some lifts in the analyses to allow gaps in time for the placement of lifts not modeled in the 2-D and 3-D grids. Mean daily air temperatures used in the thermal analyses were obtained from the National Oceanic and Atmospheric Administration Service for the weather station located nearest the construction site at Paducah, KY. Three additional provisions were specified by the Louisville District: 1) a maximum 60 °F placement temperature; 2) no insulation; and 3) a start-of-construction date of June 20. These provisions limited the scope of this investigation to construction which could be concluded prior to the earliest possible onset of freezing temperatures. By the end of the construction period, ambient temperature was approximately 60 °F. To ensure that placement temperatures remained below ambient temperatures, a 60-°F-placement temperature was used in the floor, and wall placement temperatures were varied from 60 °F to 50 °F as the ambient temperature dropped.

Thermal Boundary Conditions

The lower boundary of the foundation material was fixed at 57.2 °F, the mean yearly temperature for the construction site. The initial vertical distribution of temperatures in the foundation was computed by exposing the foundation surface to 2 years of ambient temperature variation. Calculated soil temperatures for June 20 of the second year were used as the starting foundation temperature distribution. No horizontal heat flow was permitted through vertical model boundaries at symmetric monolith center lines in 2-D analyses, at planes of symmetry in 3-D analyses, or through vertical soil boundaries. Formwork was assumed to be 0.75-in. plywood with a conductance of 1.07 Btu/ft^2-h-°F. Vertical formwork was assumed to be removed 2 days after placement of a lift. Horizontal formwork as used



Figrue 2. Analysis grids

in ceilings of culverts was removed 5 days after concrete placement. Formwork removal was simulated by changing the surface film coefficients. The surfaces of culverts were assumed to be exposed to the same wind velocity as other exterior surfaces.

Boundary Conditions for Stress Analyses

For the stress analyses, no foundation elements were included. The finite element grids were supported at the axes of symmetry with rollers and at the base with vertical and lateral springs applied at the nodes to model piles. A vertical soil stiffness of 300 psi per inch provided support during early phases of construction when the concrete still has a fairly low modulus. All piles were HP 14 × 117, 708 in. in length, vertically oriented, and equally spaced at 75 in. in the transverse and longitudinal directions. To be consistent with plane strain modeling assumptions, the vertical stiffness was modified to obtain an average pile stiffness per inch thickness of concrete. The lateral pile stiffness is dependent on soil support and pile spacing. Two soil profiles, developed from boring logs, were used to generate p-y curves. The curve providing the least displacement was selected for use in the analyses. The curve was then modified and reduced for pile group effect to obtain the average lateral stiffness. Since the p-y curve was nearly linear over the expected range of displacements, the nonlinear lateral pile stiffness was replaced using a linear spring based on a 0.2-in. displacement. At times prior to the removal of the formwork, the concrete was assumed to be supported by the formwork. Gravity, in the form of body forces, was not applied to new concrete until 2 days after placement for most lifts and 5 days after placement for lifts containing roof sections. Until that time, dead loads from new concrete were simulated as pressures on existing concrete.

2-D Heat Transfer Analyses

Analyses were performed on models for both placement methods using properties of both concrete mixtures. Mixture 11 properties were also used for an out-of-plane analysis of a strip placement method floor section parallel to the direction of flow. Temperatures and time of occurrence are provided in Table 1. In the strip placement analyses, heat flow in the XY plane was one-dimensional (1-D). The temperatures at a given time and elevation were nearly constant throughout the floor except near the boundary. Temperatures in the culvert walls became fairly uniform and approached ambient temperature shortly after placement. In the block placement analyses, the heat flow was also 1-D. Due to the lift placements, temperatures varied slightly across the floor at any given elevation. Initially, temperatures in the floor were slightly higher than in the strip method. This was due to the increased lift heights. After 50 days similar temperatures were predicted in both placement methods. The heat flow in the out-of-plane analysis was 1-D except near vertical lift boundaries. After time, all analyses predicted, as expected, temperatures that approached, then paralleled the ambient conditions.

3-D Heat Transfer Analysis

This analysis used mixture 11 properties in a block placement method model. In the plane of symmetry transverse to the axis of flow, no heat flow parallel to the axis of flow is allowed. This is the same condition that exists in the 2-D analysis, i.e., heat flow in only the X and Y directions. This means that predicted temperatures at the transverse center plane of the 3-D model should be the same as those in a 2-D analysis of the structure under the same conditions. When temperatures at the appropriate 3-D elements were compared with corresponding elements in the 2-D analysis, temperatures in the 3-D analysis varied only slightly from those in the 2-D analysis. The largest difference, approximately 2 °F, occurred at the bottom node in section 1 and was probably due to the use of larger elements in the soil. Predicted temperatures at this node converged at approximately 120 days, and temperature differentials across the floor at times prior to that were only slightly greater than those in the 2-D analysis. Maximum temperature differentials at floor sections were the same in both analyses. Observation of contour plots made at sections through the center planes and at the vertical interface indicated that the

Table 1
Temperature Comparison

	Mixture 6			Mixture 11		
Location	Max. (°F)	Max. Diff. (°F)	Time (Days)	Max. (°F)	Max. Diff. (°F)	Time (Days)
Strip Method						
Top center of floor	90	—	28	92	—	34
	—	20	200	—	26	200
Midheight of center wall	85	—	76	88	—	88
	—	9	214	—	21	109
Block Method						
Top center of floor	95	—	24	96.8	—	29
	—	26	214	—	27	200
Out-of-Plane						
Lift 3,6 interface	—	—	—	93.5	—	25
	—	—	—	—	27	200

area of 1-D heat flow along the length of each monolith extended through the center 1/2 of the monolith.

2-D Stress Analyses

Analyses included plane stress and plane strain analyses of the grids for both placement methods and both concrete mixtures and plane stress and plane strain analyses for the out-of-plane floor grid using mixture 11. Selected results are presented in Table 2 and Figure 3. All analyses of the strip placement method showed the highest tensile stresses occurred at the top center of the floor, at the top of the center wall culvert, and at the lower corner of the center wall gallery. Vertical and shear stresses in the floor were low and horizontal stresses were primarily compressive for the first 50 days. Center wall vertical and shear stresses were also low with the horizontal tensile stress decreasing over time. Analyses of the block placement method showed tensile stresses in the same regions as the strip placement method. Maximum horizontal tensile stresses in the floor were somewhat lower in the block placement analyses. This was due to less restraint against volumetric changes in the newer lifts. The out-of-plane analyses showed high tensile stresses at the midmonolith joint in the top two lifts and compressive stresses normal to the interfaces in the lower lifts. The tensile stresses tended to decrease over time. No cracking occurred in the plane stress analyses. However, cracking due to out-of-plane stresses occurred in the plane strain analyses and may have affected the accuracy of predicted stresses in some areas of the structure.

3-D Stress Analysis

Maximum tensile stresses should occur at the plane of symmetry transverse to the direction of flow. Mixture 11 properties were used in the block placement scheme for this analysis. The maximum horizontal tensile stress in the floor section was slightly less than the maximums predicted in either the 2-D plane strain or plane stress block placement analyses. In the 2-D analyses, out-of-plane stress was affected by the restraints imposed by the plane strain formulation in contrast to the 3-D analysis where stress in the Z-direction was induced by the restraint to volumetric contractions imposed by the existing concrete. The Poisson's effect on early compressive stresses lowered horizontal tensile stresses in the plane of interest. Wall stresses generally fell within the stresses predicted for the plane stress or plane strain analyses.

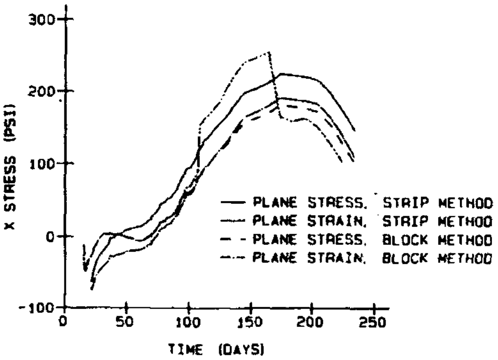
Conclusions

The following conclusions were drawn from Phase II:

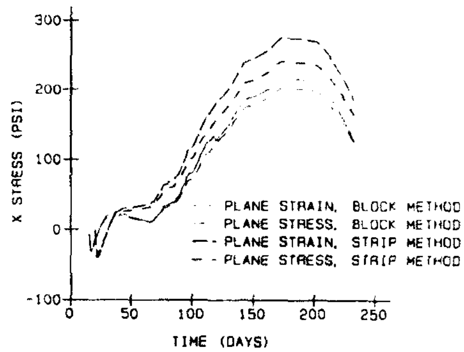
- In general, similar results were obtained from thermal stress analyses using both concrete mixtures. Mixture 11 analyses produced slightly higher stresses in the

Table 2
Maximum Horizontal Stress Comparison

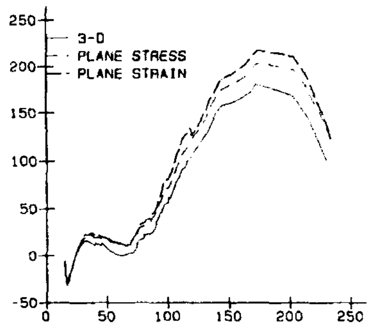
	Mixture 6				Mixture 11			
	Plane Stress		Plain Strain		Plane Stress		Plane Strain	
Location	Stress (Psi)	Time (Days)	Stress (Psi)	Time (Days)	Stress (Psi)	Time (Days)	Stress (Psi)	Time (Days
Strip Method								
Top center of floor	225	173.5	191	173.5	241	170	275	170
Block Method								
Top center of floor	182	173.5	255	163.5	203	175	221	175
Out-of-Plane								
Top lift interface	—	—	—	—	127.4	180	109	180



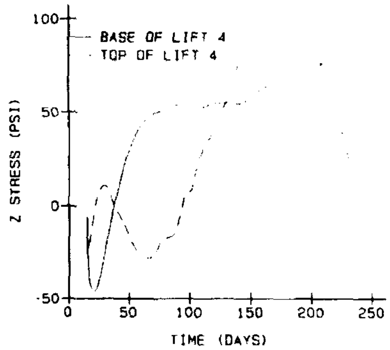
Mixture 6, 2-D analyses—
Horizontal stress at top center of floor



Mixture 11, 2-D analyses—
Horizontal stress at top center of floor



Mixture 11, Analyses—
Block placement method



3-D Analysis—
Stresses parallel to the direction of flow

Figure 3. Select stress versus time plots

floor and mixture 6 analyses produced slightly higher stresses in the walls. Both mixtures exhibited low early-time modulus and low shrinkage. However, specific creep strains were much larger for mixture 11 than for mixture 6. For a structure under tension at relatively early times, this high early-time creep is usually advantageous. Due to the low placement temperature and low shrinkage, stresses in the top of the floor were compressive for the first 20 to 30 days in all analyses. For a structure under compressive stress throughout this period, high early-time creep may act to reduce early compressive stresses, shifting the entire stress-time curve upward into a higher tension at late times. Since early-time stresses in the floor were primarily compressive, maximum tensile stresses were due to seasonal

fluctuations in ambient temperature rather than to initial heat rise in the concrete. All in-plane stresses and strains fell within acceptable ranges as set forth in ETL 1110-2-234.

- Plane strain analyses generally produced higher in-plane stresses than corresponding plane stress analyses. The appropriate type of analysis depends on the geometry of the structure. A plane strain analysis is considered to be valid when the out-of-plane length is greater than three times the in-plane dimensions. This is not the case in the "W" frame structure, and the plane stress results should be more realistic. Plane strain results should provide an upper bound for tensile stresses.

- In areas of 1-D heat flow, tensile stresses tended to be perpendicular to the direction of heat flow. Stresses in the direction of heat flow and shear stresses were negligible. Areas of 1-D heat flow included the floor, culvert walls, and the top section of the outer wall. Stress concentrations at wall openings in the center wall were due to differential displacements around the openings. These differences are a result of the variations in the amount of restraint to thermal volumetric changes provided by the concrete and the different rates of cooling for the two sides of the openings. High tensile stresses at wall openings occur relatively early after placement of the concrete (within 20 days), and may indicate problem areas for early-time cracking.

- All analyses were made using the assumption that the structure would act monolithically. For the block placement method to produce a monolithic structure, the joint between the floor sections must be capable of sustaining the maximum level of tension in the floor to prevent the joint from opening. If the joint opens due to tensile stresses at the top, the resulting crack could be propagated downward with time. Although the joint would probably not open through the entire depth of the floor, the ability of the monolith to carry stresses across the joint could be seriously affected.

- In the strip placement method, the mid-monolith floor joint is also in tension, and joints at lower elevations experience low tensile stresses at early times. If these joints do not remain closed, cracking is likely to occur. While this cracking will not prevent th structure from acting as a monolith, it can lead to maintenance problems. Cracks in the wall over the midmonolith floor joint could potentially be exposed to freeze-thaw conditions which would tend to aggravate cracking. These results indicate that the strip method is the preferable method of placement. This method locates all joints transverse to the direction of flow. Since tensile stresses in the direction of flow are much smaller than those transverse to the direction of flow, tensile stresses normal to the joints will be lower for the strip placement method.

Recommendations

Based on results of Phase II, the following are recommended:

- Both placement schemes appeared to be constructable under the conditions assumed for these analyses. The assumption of monolithic behavior upon which the analysis is based is valid for the strip placement scheme but may not be valid for the block placement scheme. Therefore, of the two placement schemes studied, the strip placement method was recommended.

- Since tensile stresses in the floor are primarily due to long-term temperature changes rather than heat rise during hydration, the requirement of a 60-°F-placement temperature in the floor may not be necessary. Using a higher placement temperature would result in a cost savings for the Louisville District and should be investigated.

- Vertical construction joints should be prepared in a manner to ensure that the expected levels of tensile stress across the joints can be maintained. If this cannot be done, consideration should be given to reducing the monolith spacing in order to eliminate vertical joints within a monolith.

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The Yazoo Basin Demonstration Erosion Control Project

by

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Abstract

The Yazoo Basin hill area predominantly consists of highly erodible and dispersive soils. As these areas have been cleared for agricultural purposes, erosion and the consequent loss of productive farmland has become a persistent problem. Further, the erosion in the uplands contributes to sediment transport problems and channel filling in the downstream reaches, resulting in the loss of flood flow capacity.

This paper provides a brief history of the development of the project and addresses the topics listed. Problems that have arisen during the progress of the project are also addressed.

The Demonstration Erosion Control Project (DEC) consists of a systematic process for solving the erosion, sedimentation, and flood control problems for a watershed and defining the needed structural measures by analyzing the hydraulic and geotechnical stability of the major tributaries of the watershed.

Meeting the project schedules has been difficult, and the need to accelerate the work has resulted in the use of innovative techniques to expedite Section 404(b) blanket permits and cultural resource evaluations as well as the use of unusual contracts for both design and construction.

Structural measures being designed as a part of the DEC include riser-pipe grade control structures (GCS), low-drop GCS, high-drop GCS, floodwater retarding structures, stone bank stabilization, channel enlargement/cleanout, and box culvert GCS. Examples of these structures and the situations in which they are used are presented. Environmental enhancements, such as reforestation, duck boxes, raptor nesting structures, etc., are also being included where appropriate to ensure a positive benefit to the environment in addition to the water quality benefits which result from the control of erosion and sedimentation.

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Project Background

Description of problem

The Demonstration Erosion Control Project (DEC) is located in the hill area above the Yazoo Delta (Figure 1). This area is characterized by steep rolling hills with windblown dust (Loess) deposits up to 200 ft thick at the western edge of the hill line and gradually thinning to the east. These Loess deposits are highly erodible. As European cultures began to settle in this area in the mid 1800's, agriculture with associated land clearing increased dramatically. The combination of the highly erodible soils and rampant land clearing resulted in heavily gullied lands and exhausted fields in what previously had been, albeit temporarily, highly productive cotton land. From 1830 to 1920, average annual sediment yield increased from 0.05-0.15 tons/acre/year to 50 tons/acre/year. The decline of agriculture in the area combined with remedial actions taken to date had reduced the sediment yield to 5-10 tons/acre/year in 1980. However, erosion and sedimentation continue to be severe problems in the area.

Legislative background

In the late 1800's, recognizing that the accumulation of silts and debris was restricting channel capacities, the Mississippi legislature passed a series of laws enabling the formation of local (county) drainage districts for drainage planning. These districts planned works for short reaches of streams of interest to the district, but overall coordination of work within a watershed was lacking. In the late 1930's, the Federal Government became involved in an effort to coordinate the erosion, drainage, and flood control work, the primary benefits being the planning on a watershed basis, the implementation of soil conservation practices, and comprehensive flood control activities. The Soil Conservation Service (SCS) concentrated on land use/treatment measures and small erosion control structures, while the Corps of Engineers worked on major flood control reservoirs. In 1974, the Water Resources Development Act, Section 32, authorized a streambank erosion control evaluation and demonstration program (SECED). This program allowed for the evaluation of stream stabilization techniques but did not allow for complete system stabilization for any watershed. Public Law 98-8, the Emergency Jobs Appropriation Act of 1983, authorized the DEC as a joint effort between the Corps and SCS to develop a comprehensive solution to the erosion, sedimentation, and flood control problems of the study area. This authority was further expanded in Public Law 98-50, the Energy and Water Development Appropriation Act of 1984. Public Law 99-662, the Water Resources Development Act of 1986, exempted the DEC from the requirement of local sponsor cost sharing. Six watersheds were initially identified. An additional 9 watersheds have been added through various agricultural bills to bring the DEC up to 15 watersheds within the Yazoo Basin hill area.

Current operating procedure

As previously discussed, this project was conceived as a joint effort among Federal agencies. Currently, the Corps, SCS, Agricultural Research Service (ARS), and Waterways Experiment Station (WES) are all involved. Of the four agencies involved, only the Corps and SCS have a design and construction role. ARS and WES assist in establishing baseline conditions, monitoring the effects of the project, and in research directed toward improving the methods being used in the project. The overall direction of the project is provided through a task force made up of representatives of the four partners. The task force assigns a particular watershed to the Corps or SCS for preparation of a work plan for that watershed. For the Corps, this is considered a supplement to General Design Memorandum No. 54 (Reduced Scope) (US Army Corps of Engineers 1990), and the SCS refers to their document as a watershed work plan. Although a basic plan for a watershed has been prepared by of one of the lead agencies, work in the watershed may be done by either agency, depending on the nature of the work and capabilities of the agencies. This may result in Corps and SCS projects being constructed side by side on the same stream.

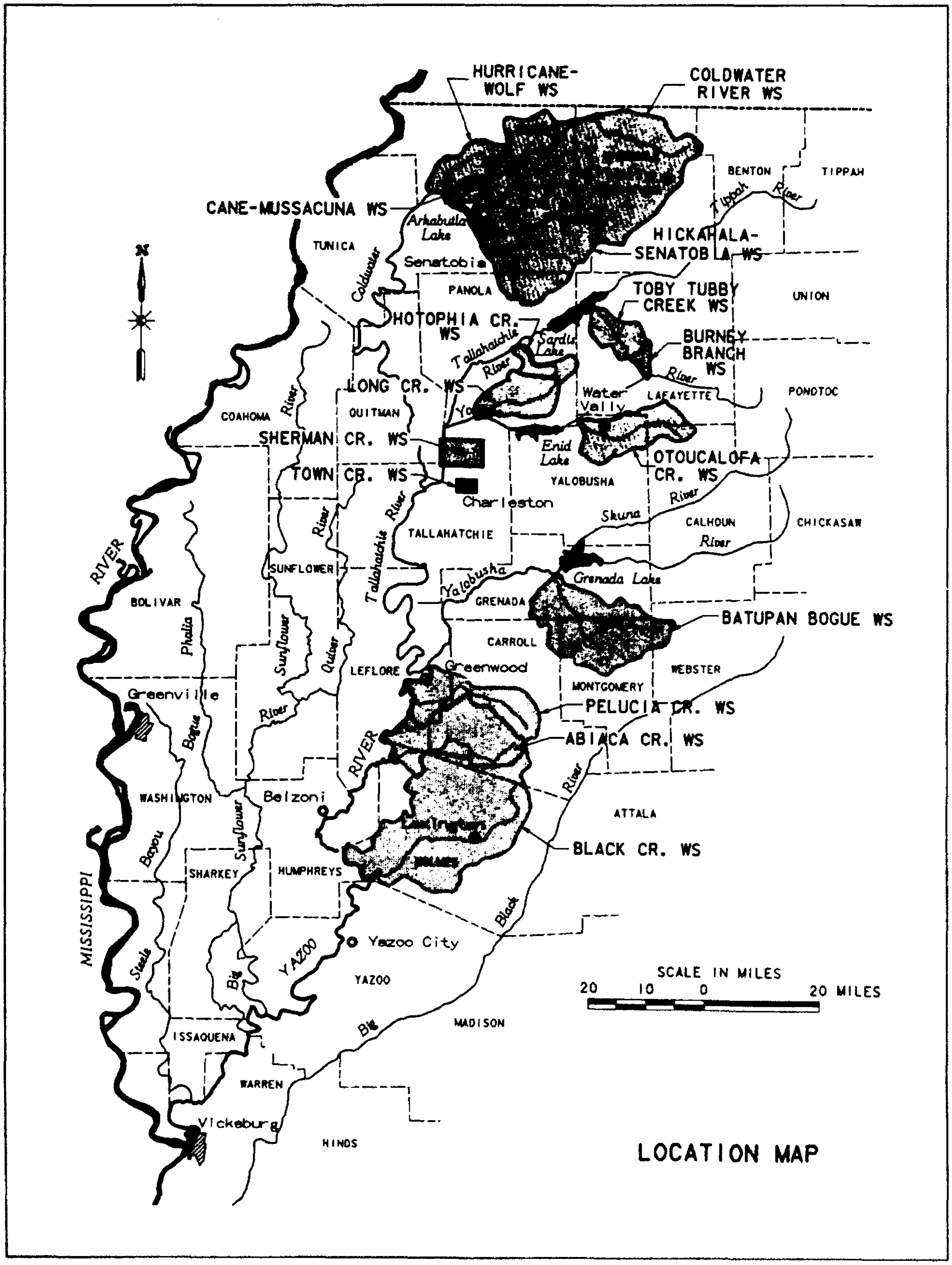

Figure 1. Location map

Cost of the project

As the DEC has grown in geographical scope from 6 to 15 watersheds, the project funding has increased significantly. Beginning in FY 1985 with funding of $6.2 million, the project was funded at $16.4 million in FY 1990. Funding is expected to increase to $25 million in FY 1993 and remain stable at that level for at least 5 years. Total project cost is estimated at $800 million, with annual costs of $10 million for operation, maintenance, and replacements. Monitoring will continue for 5 years after completion of construction, and these costs are expected to reach $14.5 million over the life of the project. Approximately 60 percent of construction will be done by the Corps, with the remainder done by SCS. All prices are for October 1989.

Table 1
Structural Features
Demonstration Erosion Control Project

Type	Unit	Total
High-Drop GCS	ea.	12
Low-Drop GCS	ea.	238
Riser Pipe GCS (Pipe Drops)	ea.	2,412
Bank Stabilization	ft.	1,492,683
Flood Water Retarding Structures	ea.	72
Debris Basins	ea.	191
Intermediate Dams	ea.	10
FWRS (Rehab.)	ea.	89
Debris Basin (Rehab.)	ea.	1,101
Intermediate Dams (Rehab.)	ea.	65
Channels	mi.	65.44
Levee (including app. str.)	mi.	24.50
Pumping Station	ea.	4
Land Treatment	misc.	$14,394,399
Bridge Replacement	ea.	53
Box Culvert GCS	ea.	79

Scope of the project

The geographical scope of the project, as previously discussed, has increased from 6 to 15 watersheds. This increased the total area of the DEC to 1,464,000 acres, about 5.9 percent of the total Yazoo Basin drainage area. Table 1 shows a breakdown of the numbers and types of structures to be constructed. Although the types of work involved are not technically demanding, managing the design and construction of the large numbers of individual structures involved is challenging.

Development of Solutions

The systems approach

To accomplish the goals of the authorizing legislation to have watershed-wide solutions of the erosion, sedimentation, and flood control problems in the area, and to avoid the mistakes of the piecemeal approach used by the states in the pre1930's to solve these problems, the DEC focuses on entire watersheds. The systems approach used in this project is more complex than can be addressed in this paper. However, an overview of the methods used is essential to the discussion of the project. The first step in the systems approach consists of identifying the type and extent of system-wide problems as well as the localized problems which must be addressed. This is accomplished through field investigations and discussions with local officials and local landowners. The types of problems which are typically identified are surface erosion, channel erosion, flooding, sedimentation, damages to the transportation and/or communications systems, and degradation of the environment. After the specific problems for a watershed are identified, data are gathered to indicate both past and present conditions in the watershed. The goal of the data gathering is to determine what changes are occurring in the watershed and at what rate they are occurring. The data gathered typically consist of channel thalweg and cross-sectional surveys, aerial photography, and other data relating to hydrologic, hydraulic, and geologic history of the watershed. Concurrent with the data gathering, field investigations are conducted so that an assessment can be made of the overall stability of the system and the rate at which any changes are taking place. For each reach of the major channels in the watershed, both hydraulic and geotechnical stability are analyzed based on comparison of the data gathered with emperical relationships which relate various physical features to stability. In some cases the

results show that a stream is hydraulically stable (i.e., slope, flow, total stream energy, sediment load, etc. are within the limits of stable parameters) but geotechnically unstable (i.e., bank heights and angles, soil strength, etc. are not within the limits of stable parameters). In other cases the reverse is true. In these cases the reliability and availability of data used in determining stability are considered. The corrective measures to be used in a particular reach of channel are then selected to restore the channel to hydraulic and/or geotechnical stability and to alleviate the system problems previously identified.

Types of Structures

Riser-pipe grade control structures

Riser-pipe GCS (Figure 2), also referred to as drop pipes by the SCS, are used to control gully erosion which occurs where runoff from areas adjacent to a stream concentrate and drop over top bank into the stream. This active erosion process contributes large amounts of sediment into stream systems. As they grow and progress, large amounts of farmland may become unusable, and access to fields may be disrupted. The riser-pipe GCS are simple, corrugated metal structures primarily consisting of a vertical riser and a horizontal conduit through an embankment which channels flow into the structure. Appurtenances to the structure include a filter drainage diaphragm for control of through seepage and to prevent piping along the conduit; an anti-vortex baffle to help maintain weir flow into the structure; a concrete foundation beneath the riser; and a top slab to prevent erosion around the inlet. For structures having a diameter of 48 in. and larger, timber pile pipe supports and stone protection in the outlet channel are added. These structures are designed to pass a design flow ranging from a 2-year to a 10-year frequency event, depending on the size and criticality of the structure. Normally, site conditions will provide a natural emergency spillway for flows above the design flow. Where this does not occur, an emergency spillway is cut to protect the structure from failure. The dispersive soils frequently encountered in the DEC watersheds make these structures particularly susceptible to piping failures. However, to date we have had only one failure attributable to this cause. At this time, both aluminized and galvanized, polymer-coated pipe are being used for riser-pipe GCS. After sufficient data have been gathered concerning the serviceability of the two pipe materials, a selection will be made as to the type of pipe material which will be used for the remainder of the DEC structures.

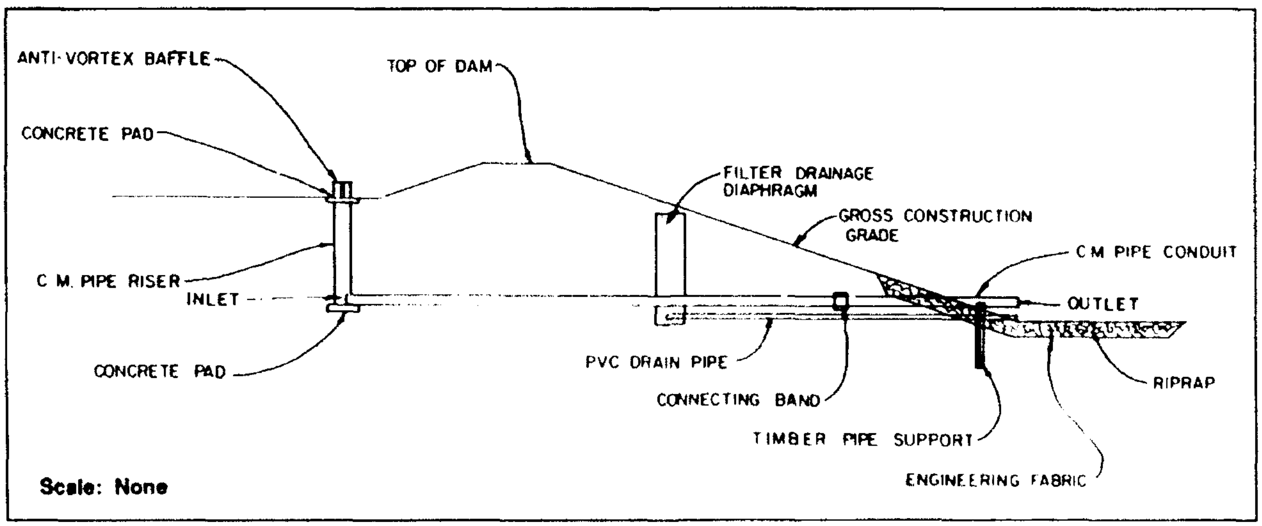

Figure 2. Profile along center line of pipe, riser-pipe grade control structure

Low-drop grade control structures

In degrading streams head cuts (areas where water drops from one level to another) progress upstream and deepen the channel. This causes bank failures which increase the top width of the channel, with the accompanying loss of land on either side of the stream. Further, bridges, water and sewer lines, and telephone and power lines can be jeoporadized. Where head cuts 6 ft or lower in height occur, low-drop GCS (Figure 3) are used to stop the head cuts from progressing further upstream. These structures consist primarily of a sheet-pile weir section, steel baffle, and riprap stilling basin. The sheet-pile weir section has steel channels bolted to the upstream and downstream faces to maintain alignment as well as a concrete cap which further stiffens the alignment of the structures and also protects the steel wales from corrosion from flowing water and sediment. The sheet-pile section is anchored with battered tension piles attached to the wales. The extensive efforts to maintain the alignment of the structure result from problems which have been encountered in maintaining the riprap on the downstream face of the weir. During high flow conditions, even extremely large rock can be displaced and leave the sheetpile with little soil support on the downstream face. The baffle located in the stilling basin consists of steel sheet-pile sections supported on vertical "H" piles braced with battered "H" piles on the downstream side. This keeps standing waves from forming and passing through the stilling basin with little or no energy dissipation. As previously discussed, loss of rock from the downstream face of the weir has been a persistent problem. Recent studies by the US Army Engineer District, Vicksburg, indicate that to ensure that dislacement will not be a problem, riprap having a top weight of over 1,500 lb/stone would be required. It is not feasible to deliver this size stone to many of the remote sites which are covered by the DEC. We are beginning to use grouted riprap on the most critical areas of the stilling basin. If, over time, this alleviates the problems associated with rock movement, the anchor piles for the weir may be omitted from future structure designs.

High-drop grade control structures

High-drop GCS serve essentially the same function as low-drop GCS but are for stabilizing head cuts 6 to 15 ft high and will generally be used in only the more critical areas of headcutting. These structures are SAF type straight-drop spillways (Figure 4). The structures primarily consist of a monolithic box-shaped weir with a subsurface drainage system and concrete head wall extensions to provide positive protection for flow around the structure. These structures are designed for the 25-year frequency flow in agricultural areas and for the 100-year flow in commercial or residential areas. Since the high-drop GCS

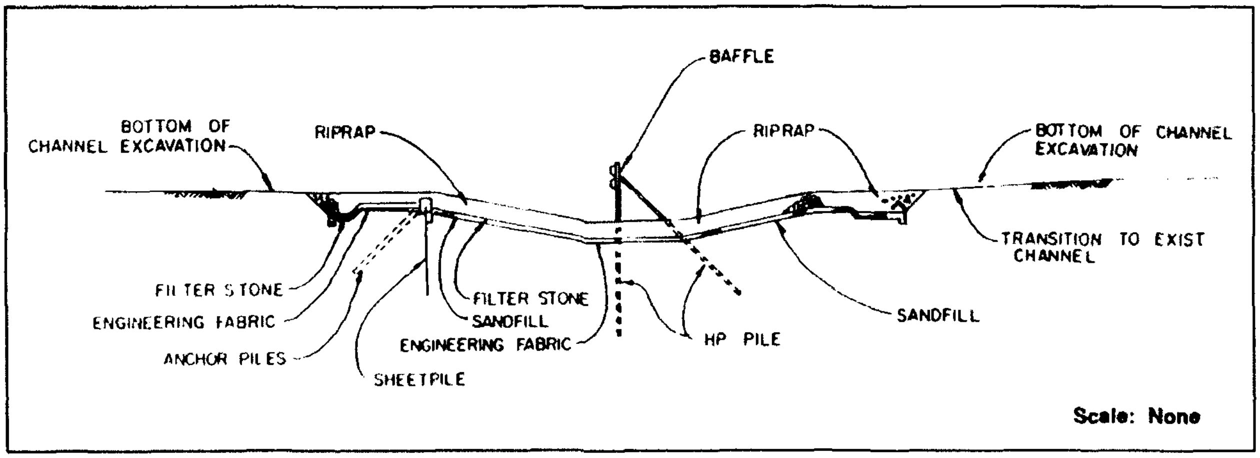

Figure 3. Profile along structure center line, low-drop grade control structure

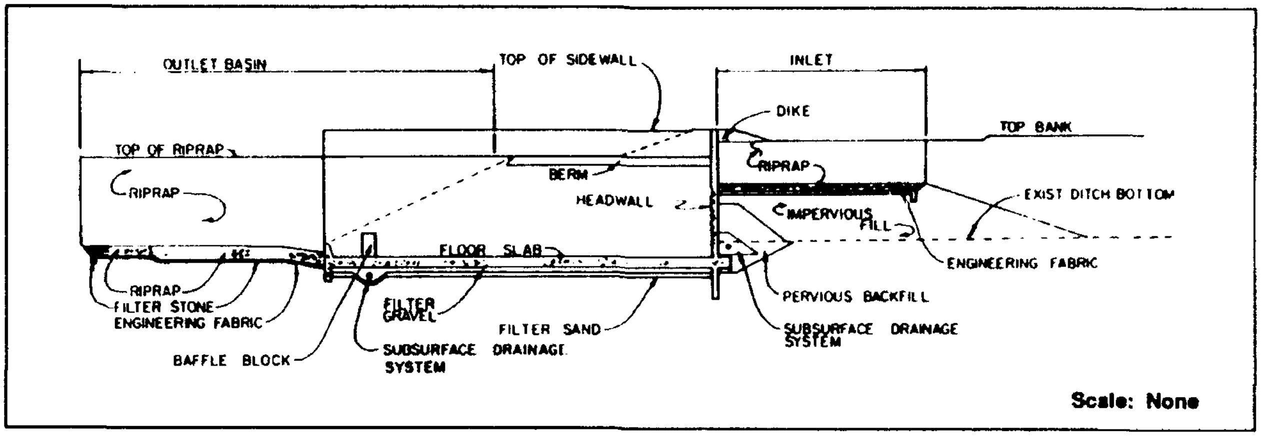

Figure 4. Profile along structure center line, high-drop grade control structure

are more expensive than most of the other DEC structures, more extensive operation and maintenance is expected to be performed. Consequently, the lands for these structures will be obtained in fee. This significantly increases the time for obtaining real estate.

Floodwater retarding structures

Floodwater retarding structures (FWRS) are small dams with uncontrolled principal spillways and grassed emergency spillway sections. The principal spillways are concrete pressure pipes supported on concrete cradles. They empty into a riprap-lined, plunge pool-type stilling basin or a concrete impact basin. These structures are used to lower the peak flow downstream of the structure and thereby slow or stop channel degradation. By lowering peak flows, the structures also provide flood control benefits and will be used where rural communities face recurring flooding problems. These structures will primarily be designed and constructed by SCS.

Bank stabilization

Bank stabilization is used to stabilize streambanks eroding as a result of scour or meandering where the stream reach is otherwise stable. The primary types of stabilization are bank paving, transverse dikes, and longitudinal dikes (Figure 5). In general, longitudinal dikes are used in stream reaches with low banks or sharp bends, or at locations where structures make transverse dikes impractical. The longitudinal dikes are tied back to the bank slope at each end and intermittently with sections similar to the transverse dikes. Transverse dikes are used in relatively straight, high-banked reaches. Bank paving is used primarily to protect specific structures which are threatened by erosion and on freshly graded or filled banks.

Box culvert grade control structures

Box culvert GCS (Figure 6) are similar in function to riser-pipe GCS. These structures will be used primarily where head cuts are threatening roads and highways. In these cases the structure embankment will serve as the roadbed. These structures will consist of a horizontal conduit which may be corrugated metal, precast concrete, or poured-in-place concrete, depending on the size of the structure. The vertical conduit would generally consist of a concrete box-type structure with the top set flush with the existing channel bottom.

Environmental features

A number of environmental features are being considered for incorporation into each structure type as appropriate for the specific site location. Among these are fish shelters and catfish nesting structures, raptor nesting structures, wood duck nesting boxes, and planting of mast producing trees and environmentally desirable forms of ground cover. In general, the structures themselves

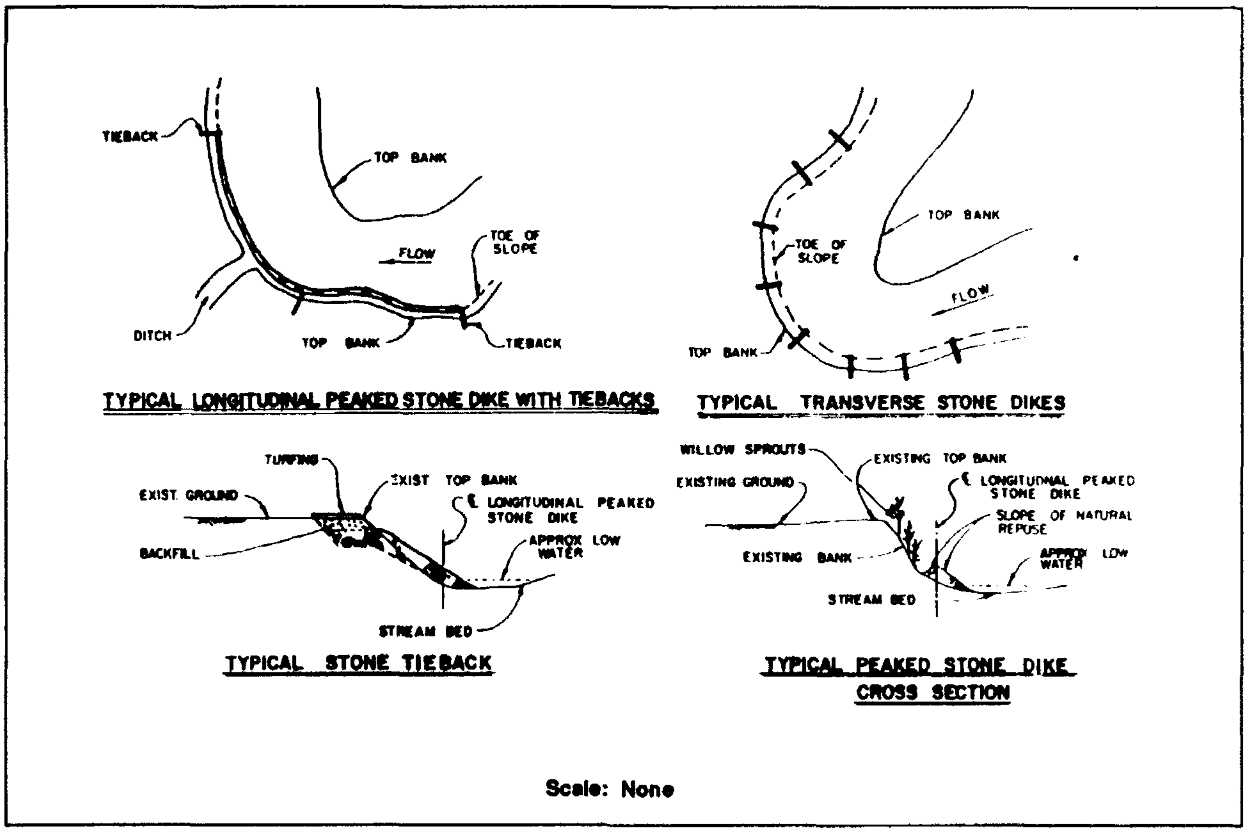

Figure 5. Bank stabilization

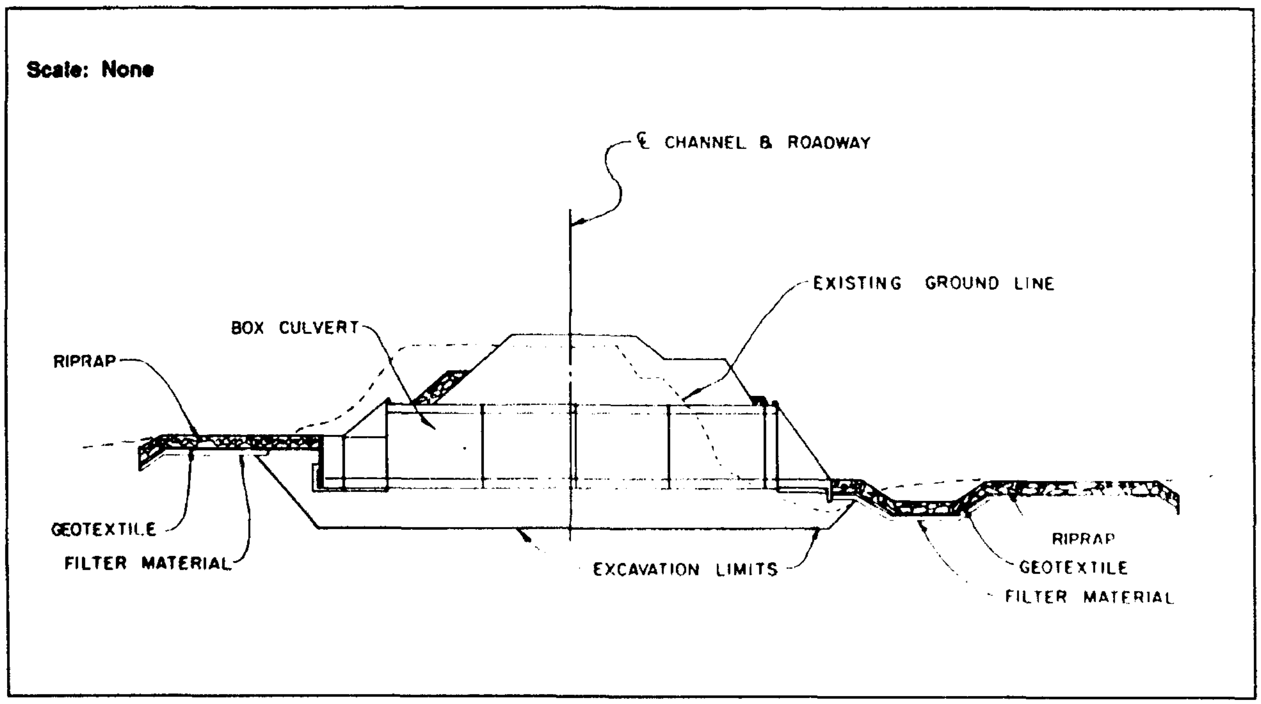

Figure 6. Box culvert and channel center line

will provide for improved water quality and enhance the environment by reducing sedimentation problems and providing more stable channel beds.

Other structures

In addition to the structures discussed, a number of other types of features will be included in the DEC. Of these features, SCS will be the sole design/construction agent for debris basins and intermediate dams; rehabilitation of FWRS, debris basins, and intermediate dams; land treatment; and bridge replacements. Channels and levees may be designed/constructed by either agency but are not addressed here since they are a relatively minor portion of the DEC. The pumping plants may be designed/constructed by either agency. A detailed discussion of these structures would be beyond the scope of this paper.

Innovative Techniques

Section 404(b) blanket permits

To expedite construction of DEC work, a general permit was processed which gives Section 404(b) approval for all riser-pipe GCS and bank stabilization work. The permit also gives approval for low-drop GCS provided the total fill to be placed in a channel is less than 5,000 cu yd. This reduces the Section 404(b) approval time for the most numerous structure types from 120 days to 30 days or less.

Cultural resources surveys

Due to the large number of structures to be constructed, a method was needed to expedite completion of cultural resources surveys. A 10-percent sample survey skewed to cover the most likely archeological areas has been done in some of the DEC watersheds and will be done in others. The survey data are being used to develop a predictive model where archeological sites are likely to be located and where National Register eligible sites are likely to exist. This eliminates the need to look at many individual construction sites.

Nonsite specific bank stabilization

To deal with a need for small reaches of bank stabilization which would not warrant a separate design, a contract was developed which identified the site, location, and type of work required and included standard drawings of each type of work. The bid items for the contract will be based on unit prices and will be tiered based on the total quantities placed. This contract allows for work at multiple small locations which would be very manpower intensive to design as individual sites.

A-E contracting

Early in the life of the DEC it was recognized that the numbers of structures to be designed would be large and that A-E contracting would be required. A contract was established for design of a large portion of the DEC. This is a multiyear contract under which the prime contractor contracts all design effort with up to 18 subcontractors, serves as a first level of review, and provides project management for the subcontractors. This gives the Corps a single point of contact for work with a large number of subcontractors and allows for design of a large number of structures with a minimum of Corps labor. Indefinite delivery-order contracts are also being used to design the smaller structures. Upon completion of the current multiyear contract, it is anticipated that several fixed-fee, A-E contracts will be negotiated to accomplish the bulk of the DEC design.

Conclusion

The DEC is an interesting and challenging project which requires involvement by a wide spectrum of disciplines in the Vicksburg District as well as in the A-E community. It offers an opportunity to use unique and innovative methods, to see rapid progress being made, and to produce projects which are useful, environmentally desirable, and which will enjoy wide public support.

References

US Army Corps of Engineers. 1990. "General Design Memorandum No. 54 (Reduced Scope), Demonstration Erosion Control Project," Vicksburg, MS.

Quadruple 84-Inch Corrugated Metal Pipe Repair

by
Tamara L. Atchley, PE[1]

Abstract

The Harrisonville and Ivy Landing Drainage and Levee District gravity drainage structure, located south of Harrisonville, IL, has the largest diameter pipes and the highest fill above the pipes of any such structure located within the St. Louis District. The original structure consisted of four 84-in. corrugated metal pipes (CMP), gated on the Mississippi side of the levee. These pipes permitted gravity drainage of the interior stormwater runoff and the flow of Maeystown Creek, which flows directly into the quadruple CMP pipes.

During the 1985 annual inspection, it was noted that the gravity drain was deformed. Settlement of the levee crown and landside toe was also apparent. In 1990, plans and specifications were initiated for the repair of the structure. Due to unusual aspects of this project, alternate methods of repair were developed and included in the plans and specifications. Although this project was cost-shared with a local sponsor, Federal participation was limited due to the particular way funding was obtained. This paper will discuss the project's history, including the innovative design methods and materials as well as the unusual aspects of the project funding.

Introduction

There are a number of gravity drainage structures within the flood control system of the St. Louis District (SLD). These structures have generally used corrugated metal pipe (CMP) to provide drainage through the levee. Most of these conduits have been in service for many years, and a number of them have developed pipe-related problems. Often soil support around the pipe is lost by means of soil infiltration at the pipe joints. The eventual result is pipe failure.

The Harrisonville and Ivy Landing gravity drainage structure, located south of Harrisonville, IL, is such a structure. The project is located in Monroe County, IL, on the left bank of the Mississippi River between river miles 141 and 156 above the Ohio River. The structure was part of an original project to upgrade the existing levee system. Construction was completed for the structure in 1948. The structure consisted of four 84-in. corrugated metal pipes through the levee. Each pipe was approximately 333 ft long and was gated at an outlet headwall on the Mississippi side of the levee. A concrete apron was located at the outlet. There was no inlet section; the pipes were beveled to match the slope of the levee at that point. At the levee crown, approximately 36 ft (+/-) of fill was above the top of the pipes.

During periods when the river is low, these gravity drains provide for passage of water from Maeystown Creek from the protected

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area to the Mississippi River. During high-water periods, the sluice gates are closed, and interior flow is stored or diverted to the Maeystown Pump Station (located adjacent to the gravity drainage structure) which pumps it into the river.

During the annual inspection of completed projects in 1985, it was noted that the gravity drain was deformed, causing settlement of the levee crown and landside levee toe. The Harrisonville and Ivy Landing Drainage and Levee District No. 2 (the local sponsor) was notified to make necessary repairs. During successive inspections, it was found that the repairs had not been performed and that the pipe deterioration had progressed. In January 1987, the subject drains were inspected in response to a request for rehabilitation assistance as a result of the October 1986 flood of the Mississippi River. The Levee District was advised that since the previously identified repairs had not been accomplished, repairs could not be made using Public Law 84-99 Rehabilitation funds.

The inspections found that due to infiltration of soil materials at the pipe joints, each pipe had experienced a loss of structural support. The result was various degrees of collapse of the joints as well as the deflection of the pipes to an out-of-round condition. Pipe collapse resulted in isolated settlements of the levee (see Figures 1 through 4).

Since the time of the inspections, Federal authorization and funding were obtained through the Energy and Water Development Appropriations Act, 1990, to fund $1,000,000 ". . . to initiate and complete the one-time repair and rehabilitation . . ." for the project. (This amount was later reduced by Graham-Rudman.) In addition, the Illinois Department of Commerce and Community Affairs funded the project with $400,000. Any funding requirements above these amounts were the responsibility of the local sponsor (the Levee District).

The levee protects 42,884 acres, over 1,000 residents, and a $5 million dollar pump station. With these at risk from the next major flood event, the St. Louis District Engineer urged that the project proceed toward construction with the tightest possible schedule. With funding in place, plans and specifications were initiated in early 1990.

Alternative Plans

Various plans were evaluated for the repair or replacement of the damaged gravity drains. Several options of various pipe materials, diameters, and number of pipes were evaluated in order to maintain the as-built capacity of the existing quadruple 84-in. CMP. The following alternatives were the only ones that resulted in hydraulic capacities equal to or greater than the as-built quadruple 84-in. CMP.

- Four 72-in. polyethylene or reinforced fiberglass pipes with formed end sections to improve inlet conditions.
- Three 84-in. reinforced concrete pipes with cast-in-place inlet and headwall or precast end section to improve inlet conditions.

Four alternative repair plans were studied by SLD. The two most feasible plans are briefly described below. Plans 3 and 4, which involved construction of a structure in a new downstream location, were disregarded due to their high estimated construction costs. All plans included replacement of the gates, frames, and operators if funds permitted.

Plan 1 - slipline existing CMP pipe

Plan 1 involved completely relining the existing four 84-in.-diam CMP drains with four 72-in.-diam structural fiberglass liner pipes. Because of the degree of deterioration of the pipes, each of the existing lines required remining in the areas where the collapsed portions prevented the liner pipe from being pushed through the CMP. Pressure grouting was planned for the annular space around the new pipe, as well as for any voids in the soil around the intact CMP.

Figure 1. Subsidence of levee road

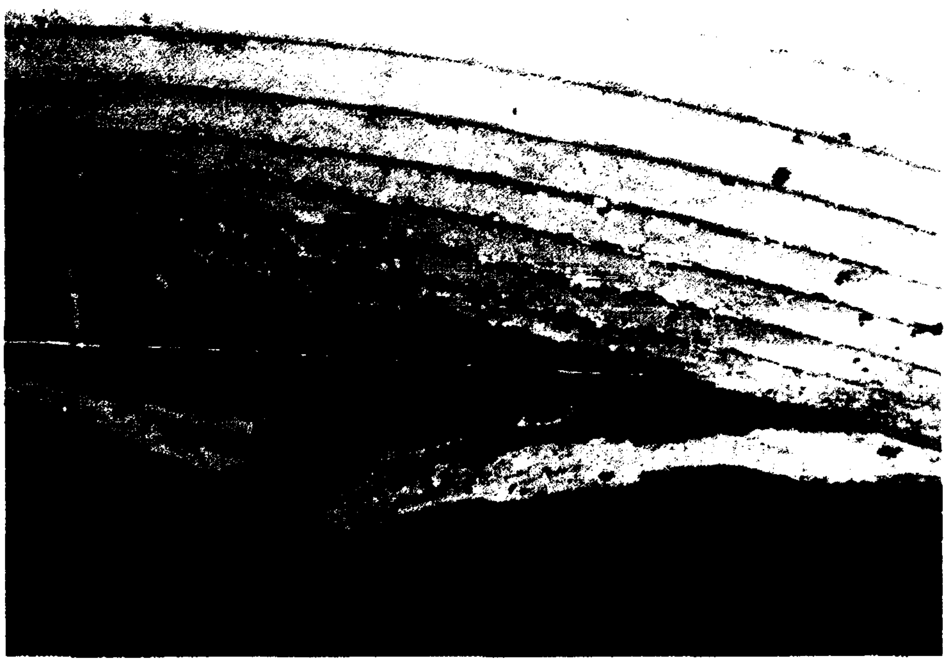

Figure 2. Opening of joint, looking upstream

Figure 3. Failed joint with soil infiltration

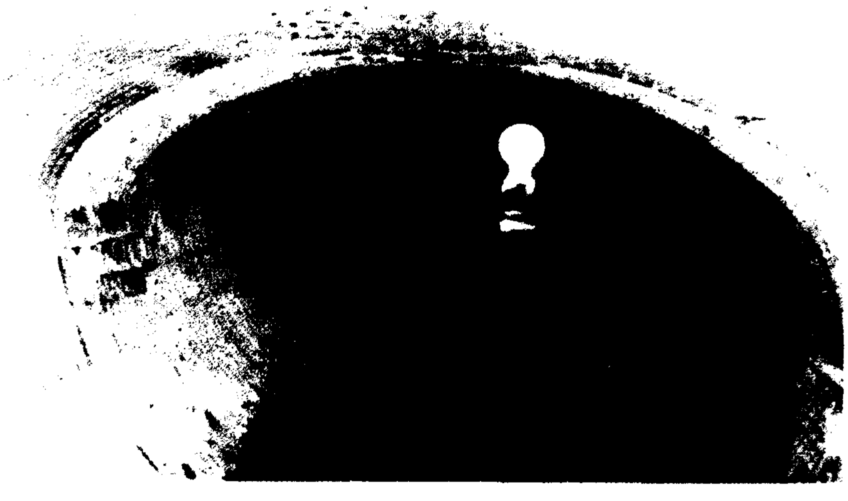

Figure 4. Out-of-round condition of CMP pipe

A bulkhead was to divert flow from Maeystown Creek away from the pipes currently under construction. Excessive flows would be regulated by the pump station.

Several disadvantages were inherent to this plan. The construction cost was heavily dependent on the length of pipe which required re-mining. Due to further deterioration of the pipes, this was subject to change prior to the start of construction. In addition, this alternative required the expertise of a specialty contractor, which could limit competition in the bidding process. Another problem was quantifying the amount of grout required to fill voids outside the CMP.

Plan 2 - replace existing CMP pipe with reinforced concrete pipe

Plan 2 replaced the existing four 84-in. CMP drains with three 84-in.-diam reinforced concrete pipes (RCP) by means of open excavation. The new concrete pipes would be attached to the existing headwall at the outlet.

With this plan, the pump station would be required to handle the interior flow. Depending on the amount of rainfall and the construction duration, the pumping cost could have varied considerably. A portion of the levee road was to be disrupted and rebuilt under this plan.

Evaluation of plans

All plan cost estimates exceeded the Federal and State funds available, even without replacing the gates. Although the basic repair cost for Plan 1 was slightly lower, the two plans were less that $200,000 apart. Because the cost estimate for Plans 1 and 2 were so close and to encourage a large number of bidders, the possibility of advertising multiple plans was discussed. SLD was informed that the local sponsor wished to proceed with Plan 1 (relining the existing pipes with fiberglass pipes). They wanted to include the gate replacement as an option if funds permitted. They did not object to advertising multiple plans, but wished to avoid any contractual or legal problems this might present. Based on their response, SLD pursued the idea of advertising more than one plan.

Another unusual aspect of this project that entered into the decision process was that due to the nature of the funding for the project, the local sponsor bore the risk of all cost increases.

A field trip was arranged with additional contractors. Following the field trip, it became apparent from discussions with contractors that a fifth alternative existed. The most severely deformed portions of the pipe (mainly located on the landside of the levee) would be open excavated and replaced. The riverside ends of the pipes, which were in much better condition, would be sliplined. This method of repair would eliminate some of the risk associated with Plan 1. This repair method would require, however, expertise in sliplining as well as open excavation construction.

Due to the uncertainties and risks associated with the various plans, it was determined that a "performance-type" specification would be developed. The end product was specified, and within certain controls such as requirements for flood protection and dewatering, the contractor was free to determine the best method of achieving the end product, whether that was relining, open cut excavation replacement, or some combination of the two.

In order to provide a common ground so that all bids could be evaluated on an equal basis, a final product of four 72-in. fiberglass pipes was specified. Fiberglass was eventually chosen after researching RCP, polyethylene, and fiberglass pipes. CMP was not considered since Appendix C of EM 1110-2-3104 does not allow use of CMP for pipe diameters larger than 60 in. or for levee embankments more than 12 ft above the pipe invert. Initially, care of interior flow was to be the responsibility of the contractor. However, because the relining alternative had an advantage over the other plans with respect to care of interior water and because the cost for this item was affected by many variables, a

separate contract was eventually arranged with the local sponsor to provide pumping of the interior flows regardless of the method of construction selected.

Local Cooperation Agreement

While efforts were being made to expedite preparation of plans and specifications for this project, efforts were also being made to prepare the Local Cooperation Agreement (LCA) with the local sponsor. On 7 June 1990, the LCA was sent to the Lower Mississippi Valley Division (LMVD). The following day, a request for a waiver allowing SLD to advertise, open bids, and conduct pre-award activities was sent to LMVD. This request was denied in Washington. Plans and specifications were sent to the SLD Contract Division on 22 June as scheduled. The Contract Division, however, could not act on the project until either the waiver was approved or the LCA was signed. Around 1 August 1990, SLD received word that SLD's District Engineer would be given the authority to sign the LCA. New advertisement and bid opening dates of 14 August and 13 September, respectively, were set. This was an approximate 6-week delay in the schedule.

Successful Bidder's Plan

Bids were opened on 13 September 1990. Six contractors submitted bids on the project in the range of $1.04 M to $1.7 M for the pipe repair only. The low bid, which was for open cut pipe replacement, was the only bid that was less than the Reasonable Government Estimate plus 25 percent.

Construction of Project

Due to the past experience with pipe joint problems in this type of structure, the specifications were written to try to ensure effective joints. The joints were required to withstand 20-psi external hydrostatic pressure and to allow no infiltration or exfiltration of soil fines through the joint. In addition, a hydrostatic leak field test was required for each joint. The joints were to withstand 30-psi internal pressure for a minimum of 1 min. The pipe itself was to be designed by the pipe manufacturer according to site-specific criteria included in the specification.

The specifications stated that the levee was to be reconstructed of material from the required excavation or borrow material (CL - lean clay, CH - fat clay, ML - silt, SC - clayey sand, and SM - silty sand). Each material specified was found to be present in the levee as evidenced by borings taken during March 1990. The specification specifically did not allow the use of an SP (sand) soil in the levee.

After bid opening but prior to award, a telephone conversation was held between SLD personnel and a technical representative of the pipe manufacturer. During this conversation, the manufacturer's representative expressed concern regarding the use of fine-grained backfill. Normally, a fine-grained backfill would not be allowed for this pipe because of problems in attaining the required compaction. He suggested the use of a sand backfill with 5-ft clay cutoffs located every 50 ft along the length of the pipe. Since this type of construction would be a problem for levee construction, it was not allowed. Per requirements in the specification, design calculations were submitted for the pipe. The calculations were originally based on the levee being entirely reconstructed of SM material. This was not approved since the prime contractor was planning on using mainly CL material from the required excavation. Revised calculations based on fine-grained soils were required. The pipe manufacturer had also proposed bedding the pipe for its entire length with an SM material. The Corps rejected this bedding method because it would compromise the integrity of the levee. The Corps required the pipe to be bedded in CL material on the riverside and landside of the levee. These clay plugs were kept within the levee berms so that the cover in these areas did not exceed in height that provided in the pipe manufacturer's letter referred to below.

After award of the contract late in September, construction proceeded from landside toward the river. Staged construction proceeded so that approximately 200 ft (about 2/3 of the levee) was excavated. The remaining riverside portion of the levee (and stockpiled material from the excavation) formed the flood protection. The second stage of construction was the remaining portion of the levee. SM material was used as fill in the middle of the levee, except that when clean sand (SP) was encountered in the pipe foundation area, it was over-excavated 1 ft and replaced with a clay (CL) material. Uniform compaction was achieved per normal construction standards, generally around 95-percent SPD or dry density of 100 pcf.

In a letter from the pipe manufacturer to the contractor which accompanied the revised calculations (received after construction had started and backfill was being placed), it was stated:

> Although the planned pipe zone backfill of silty sand (SM) is not listed as acceptable in our product brochure, it can be an adequate material to support the pipe, if a high (95 SPD) density is achieved . . . . if the required densities are consistently achieved, the planned pipe zone backfill of silty sand (SM) will result in an acceptable pipe installation (i.e. maximum initial pipe deflections of 2.5 percent and less).

In addition, the letter stated with regard to riverside berm installation:

> Backfill of the pipe zone with silty clays and clayey silts is acceptable under the following conditions:
>
> - The actual soils to be used in the pipe zone must have a liquid limit of less than 50.
> - Compaction must be minimum:
>   - 90-percent SPD (55-percent maximum relative density) for covers up to 10 ft.
>   - 95-percent SPD (70-percent maximum relative density) for covers up to 16 ft.
> - Initial pipe deflection at full cover must be less than 2.0 percent. (Clay and silt backfills will allow some creep with time.)

Work proceeded until late December, when poor weather delayed construction. By this time, with 200 ft (+/-) of the levee open from the landside, one line had approximately 200 ft of new pipe placed. A total of seven sections (pipe section lengths of approximately 20 ft) remained to be placed for the other three pipes to complete 200 ft of new pipe in each line. The degree of backfill varied.

The pipe joint used for the project consisted of a gasketted collar, installed to one end of the pipe section at the factory. The in-place pipe joints had been unofficially tested as they were installed. The vast majority of the joints passed this internal pressure test.

Work resumed in mid-January. The official testing of the joints in the first 200 ft (+/-) of the placed pipe was started on 6 February. As required in the specifications, a representative from the pipe manufacturer was present. Several of the joints did not meet the test criteria, even though most of the joints had passed the test when testing was done as the pipe was laid. In addition, apparent offsets were observed at several of the joints; this was more evident near the inlet (landside) end. Some of the joints that looked acceptable and had little offset did not pass the joint test.

In late February, the pipe manufacturer furnished information not previously submitted which required very tight tolerances with respect to the joint installation. This information indicated that in order to not exceed the angular deflection of 1 deg as allowed for this size pipe, an offset at the joint would have to be less than 1/4 in.

After all the joints had been tested, 15 out of 61 (approximately 25 percent) failed to pass the internal pressure test; some of the pressures were as low as 3 to 6 psi. In addition, 27 joints experienced water infiltration. (Some of these joints also failed the internal pressure test.) This occurred with a groundwater level only

6 in. (+/-) above the invert elevation of the pipe. Each joint was examined, and the joint characteristics were recorded in order to determine a relationship between these characteristics and the quality of the joint. No correlation was found by Corps personnel.

The pipe manufacturer felt that the joint problems stemmed from nonuniform compaction in the bedding material. The manufacturer's definition of "uniform" compaction, however, exceeded tolerances of standard industry practices. The Corps and the prime contractor felt that the compaction required by the pipe manufacturer was unattainable in this type of construction.

The pipe manufacturer's representative recommended repairing the joints by overlaying the joints with strips of fiberglass material. Because of the seemingly random nature of the joint problems and because any joint repairs required in the future would require construction of a cofferdam to divert water from the affected pipe, it was decided that all 61 joints should be repaired with the fiberglass overlay. In this operation, resin was applied to strips of woven fiberglass as well as fiberglass mesh. The joints were then overlaid with these strips. Ten layers of the fiberglass material were applied to each joint.

Recommendations and Conclusions

Aside from the problems associated with the pipe joints, the fiberglass pipe had several favorable characteristics. It was lighter than RCP, and as such, could be handled in longer lengths (20 ft +/-), requiring fewer joints than for RCP. The flow characteristics are such that the pipe diameter could be reduced from the original pipes.

However, the joint problems encountered are a major concern with use of this pipe in this application. SLD construction personnel feel that the pipes were properly installed. Testing of the foundation by contractor and Corps personnel indicate that the foundation was properly placed and well compacted within limits typical for levee construction and pipe installation. The very tight tolerances required for the joint installation (furnished after joints had been installed) are apparently too rigid for this situation.

In view of the SLD's experience with this pipe and joint, this system may not be appropriate for use in clay levee applications unless a suitable joint is developed.